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OF THE

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Beekman C. Little, President, 1920–1921

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AMERICAN WATER WORKS ASSOCIATION

The Association is not responsible, as a body, for the facts and opinions advanced in any of the papers or discussions published in its proceedings

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WATER WORKS EXPERIENCES[1]

By Beekman Cox Little[2]

Having been actively engaged in water works affairs ever since leaving college almost thirty years ago, and having less than thirty minutes to tell of my experiences, possibly some details may be accidentally left out. Once before, when entering a hall where I was to read a paper, I was questioned by an outsider as to what was going on. When he was told, he asked what it cost to go in. I told him "*Nothing*," and he then asked, "Is it worth it?" Since then I have tried to avoid being put in a similar position. However, when your Publication Committee wants anything, they should have it, and this paper, therefore, is a result of such a request and an acknowledgment, though an immensely inadequate one, of the great work which this committee, together with the editor, are doing in producing our Association Journal. If any of you are neglecting to read this bi-monthly you are passing up one of the pleasantest experiences that comes into the life of the water works man. Outside of the benefit derived from learning what others are doing and discovering in all phases of the water works business, the Journal is good reading. You can find in it tragedy, humor, description, history, politics, mystery, and even religion once in a while.

[1] Address at the Montreal Convention, June 22, 1920.

[2] President, American Water Works Association; Superintendent, Bureau of Water, Rochester, N. Y.

Don't, I pray you, when the postman lays it in your desk, glance at the table of contents hurriedly, figure that there is nothing of particular interest to you just then and throw it aside to be forgotten. Take it home with you, send your wife and children to see Mary Pickford, and then read it carefully from cover to cover. Try this prescription and you will feel like a new man. You will go down to the office next morning with more new ideas in your head, with more ways of spending money and improving your plant than you've had in a year.

This particular burst of enthusiasm arose from a perusal of the January issue of the JOURNAL. This number certainly proves my statement,—that there is a splendid variety of interest in it. I defy anyone to read, "Meter Practices at Terre Haute" and not be glad to be alive and proud of our associate out there. Why, it is as good and as happy as Pollyanna, that drama of gladness. He calls his assistants, *Mr.* Meter Reader and *Mr.* Clerk, and insists on their knowing and practicing the Golden Rule. He always speaks of the consumer as his "valued customer." He has a printed blank thanking them when they pay their bills and another which gently chides them when they don't. In either case, he says he smiles, and keeps on smiling, and it works too. He has them hypnotized, for some of them pay, by his estimate, three or four months' meter bills in advance, as he tells them it will save them making out several small checks, and he will refund if it turns out that they have paid too much. Evidently out there they all love and trust their Water Works Manager, and it was a natural mistake when a small child said she heard some English people singing, "God save R. Gwinn."

If you have good nerves and want excitement, read the report on "Sanitary Drinking Fountains," and meet the Dancing Bacillus Prodigiosus, who, according to the report, is introduced by a pipette or with moistened lips, which sounds rather wicked. For some real thrills, live history and a splendid narrative, read the "Water Supply Problem in a Combat Division."

This January issue also attempts to do the impossible, which is always interesting, that is, to give us too much Johnson. He appears three times on the title page—has an interesting paper telling how much drinking went on in the army camps, referring only to water, however. He has an editorial praising an admirable paper on the "Water Supply at Camp McClellan," and then in a second editorial dissects for us another paper on "Ozonization,"

which is so full of dielectric and inorganic and oxidizable substances, that I believe the author and Colonel Johnson are the only two men that understand it anyway. Then he even appears in the advertising pages where our Secretary states that there are still a few copies of the "Typhoid Toll" for sale. Let me state that it would be an everlasting regret to me if my intended good-humored, though frivolous remarks were taken as a flippant and slurring criticism of either the subject matter or the authors: rather they are intended to instil curiosity and investigation on the part of those who haven't read this issue and to impel a second reading on those who have. And if any water works man has not in his library Johnson's "Typhoid Toll," he should order a copy instantly. I venture to say that very few more valuable papers have ever come forth from any national society of any kind.

I have of course touched on only a few of the good things in this copy of the Water Works Journal. With one glaring exception every article, report and comment is very much worth while. You can easily pick out that exception for yourself and I regret to say that it might have been done much better.

So much for our Journal; it is one of the bright spots which appears in our business and the water works man can easily absorb all the bright spots which come along.

Once I remember in our town the Vaudeville Theatre Manager called me up and asked about the temperature of our water, and was it too cold for the Six Dare-devil Diving Girls, and would I come up and witness the first performance? Would I?!! We strive to please, and the water was not only just right for that performance but for all of them. People are different, however. Later that same year the swimming tank of the Young Woman's Christian Association developed some complaint, algae, or red water, or a noisy meter, I've forgotten what, but although I spent some time in remedying the trouble, I never found out from personal observation how those Young Christian Women stacked up with the Diving Dare-devils.

Many queer complaints come to the office. Not long ago a woman complaining over the telephone about a high water bill, said it was an outrageous and impossible charge because she had not washed any for the past two months—until she explained that she meant—no laundry work! I agreed that it was outrageous.

The oddest problem, however, came up when a customer objected to having hot water served to his house by our Water Bureau. I told him that he must be mistaken, that if the water from the cold water pipe was hot, his pipes must run in some way too close to the furnace or kitchen range, but no, he had had no furnace fire for several months and his kitchen range was a gas stove, and that there was no way of accounting for the cold water faucets giving forth hot water. On investigation we found part of his statement true. Warm water was issuing from any faucet we turned on and the service pipe where it entered the cellar was warm to the touch. There was a way to account for it, however, as we discovered after some little difficulty. Across the street from the house was a large brewery and it seems that this plant was discharging exhaust steam directly into a sewer, and this water service pipe when it crossed and touched this steam sewer became so warm that the water was heated and no cold water could be had in this house. The brewery, upon notification found a different point of discharge for its steam, and everyone was satisfied. Now that the brewery is tamed and is manufacturing ice cream, we may expect, I suppose, a complaint that ice water is flowing from the hot water faucets.

When the Prohibitionists knocked the breweries in the head, they also, strange to say, hit us water purveyors a pretty hard blow. In Rochester, not especially known as an extensive beer-making community, our water revenues in 1920 show a loss of about $1000 per month, from six breweries, compared with their bills before the nation went dry. It would be interesting to hear the "before and after" tales of such cities as Milwaukee, St. Louis and Cincinnati, and learn how their water companies weathered this storm. While prohibition was bad enough, the great war of course was worse in a financial way to water works plants. We water works men were among the few, evidently, who did not profiteer. We have kept watching the soaring prices of other things, shoes and clothing, bread and butter, house rents, cast iron pipe, valves, chemicals, coal and drinks of all kinds excepting water, but only lately has there been a general agitation for raising water rates throughout the country. This rate raising *must* come, however, otherwise the private water companies face bankruptcy and the municipal companies only fool themselves into a false position which some day will stagger the tax-payers who will have to foot the bills.

When you come to consider the changing of *your* rates, remember again your WATER WORKS JOURNAL. Here you will find thorough discussions of this subject and many practical and valuable suggestions, and workable theories, evolved by the best authorities. *Don't* let your water boards of commissioners or aldermen arbitrarily *guess* at the proper amount of water charge to be paid, when you can get so much correct information so easily and a guide which will lead you to a much surer and more permanent solution.

While the war hurt us in many ways, there were several ways in which it helped. Many a water works manager, who for years had unavailingly advocated the use of meters, suddenly found a great ally in the exorbitant price and the scarcity of coal. The need for coal for pumping purposes could be lessened immensely by a decreased consumption of water, and meters would bring this lower consumption about. So that now there are many advocates of meters among social, business and political circles where formerly there were only bitter opponents of the system. It has even reached the point where state legislative bodies are considering making the meter system compulsory in order to prevent the great waste which prevails without it.

Yet the meter man must also see the lesson in this. Should the price of meters get excessively high is there not the possiblility that we will find some way also of lessening our need for these measuring devices? Perhaps we cannot get a substitute for meters, but why not some method of grouping a number of similar consumers under one meter, or experiment with a system of metering in rotation for certain test periods long established consumers, and average their bills, thus making one meter do again for perhaps ten or a dozen consumers?

The same principle holds true with other water works materials. High prices alone may not do it, but unreasonable or unfair prices surely will cause us to find a way of doing without or at least, of doing with less.

In trying to keep down our expenses, care must be exercised to see that our economical experiments do not turn out to be unwarranted extravagances.

If you are considering the substituting for the bell and spigot cast iron pipe, a pipe of another material or another pattern, or changing from calking lead to a metallic mixture which requires no

calking, you must use extra precautions in your tests and inspections and endeavor to have no lowering in your standards of ultimate results. The same must be true of all expedients you try.

If you are going to do away with the wiped joint and use one of the patented couplings or install a lined service pipe instead of the all lead service be sure you get the best of each thing with the strength and quality of the thing that you are giving up, and then use it properly and according to the best established custom. Also get advice from those who have gotten results from these other methods. Do not be over-enthusiastic over the apparent great superiority of the newly tried thing, nor downcast over its seeming failure at first. In other words, use your head, and when you try an experiment give it a good fair trial.

These days of high prices give, I think, the salesman of the "little-out-of-the-ordinary" article a better chance, and this is a good thing both for the seller and the prospective customer. Certainly the salesman of any water works article should get a good deal of useful information from contact with water superintendents, and I know that we can get a lot of helpful knowledge from most any salesman. If you don't believe this try it on the next one that calls on you. Ask him some questions about the cities he visits—what water bureau has the best accounting department, what one keeps the best meter records, what repair shop has the best equipment, what superintendent is particularly interested in leakage surveys, what one is a good politician, and what one is a good practical man. You may not get correct answers of course to all your questions as there are all kinds of salesmen. I even had one come into my office with spats and a cane, and he got away with it too. We naturally have *them* all ticketed and placed, but don't forget that by the same token these salesmen have us all sized up and our failings and shortcomings are not minimized as they talk us over. So for you own sake, if not for theirs, give them welcome. In another article I hinted at the value of these salesmen to the Association. I seldom meet one who is not an enthusiastic and loyal rooter for our organization, and many of us were led to become members of it through these joyful travelers. The three great factors which keep us superintendents in touch with our jobs and with each other through the years are the conventions and meetings, the JOURNAL, and the traveling salesman, and I feel that the last named easily does his third share of this work.

To the younger men who are traveling for the water works manufacturers I would suggest that they cultivate and increase this enthusiasm. Get your man interested in the Association and tell him what other members of it with whom you come in contact are thinking and talking about.

Always when occasion offers, I invite any particular salesman who happens to be on hand, to go out with me if my presence is demanded by any one of the hundred happenings which call me away from the office. It may be a tour of inspection of the water shed, a break in a large main, a big fire, about which a superintendent should always be informed, or just some everyday construction work; and on these trips I find the salesman good company, not in the way, and frequently of help with advice and suggestions. And surely my guest, if he keeps his eyes open, must gain something useful himself from this contact with the management and practical operation of the water works business; something which must be of benefit both to his employer and himself.

Not alone from these representatives of our associate members, but from correspondence and from our engineering journals we learn that all over the country there is a great let up in new construction work. Anyone knows that every water works manager deserves a rest, but long years of experience have taught me that he never gets one, and that when seemingly a chance like this for one appears, then is the time for him to dig in and be busier than ever; for it gives him an opportunity to devote his attention to overhauling, repairing and bettering his whole plant. Hydrants can be painted and repaired; stop-valves relocated and tried out for operation; mains tested for leakage; pumps and engines gone over, and new efficiency figures and pump slippage obtained. Meters can be removed for tests. It may be a good time to flush, or better yet thoroughly clean out some of your mains. We have had excellent success with contractors who do this work.

Always proud of the quality of our water, I confess it is a strain on that pride to see the actually black water and mud and rusty tubercles that flow out ahead of the go-devil or piston which is forced through the main. Very soon, however, the water clears up and the result is always a much better flow and increased pressure. The fire insurance men approve of this work and the contractors don't exaggerate in their advertising claims, and oftentimes this cleaning of a main will delay for several years the apparent need of a larger pipe.

The mention of fire insurance makes one think of the fire service connections. Don't neglect, whenever you have any time, to look after these connections and the check valves on them. If the sprinkler or fire system is connected up with a secondary, contaminated, or even suspicious supply, my experience would say—throw away the check valves and cut off the supply. There are check valves which will work, and methods of inspection which prove the functioning of these valves, but no valve and no system of inspection will prevent the trying experience that we went through some years ago. Very briefly stated the occurrence was as follows. A lift bridge over a canal in Rochester was so constructed as to be operated by either of two water systems; one, our domestic supply and the other our entirely separate auxiliary fire system of non-potable water. The two systems, however, were never intended to be turned on and into the operating house at once. The street gate on one system or the other was always shut, but as an additional precaution check valves on each system were installed. Between inspections, however, an over-zealous, ignorant canal employee obtained secretly a gate wrench, closed the open street gate and then proceeded to unbolt the flanges on the check valves in the basement of the operating house and remove the checks or flaps entirely. He then bolted the flanges on again so the valves appeared to be in order and opened *both* street gates. The result was that contaminated water entered our domestic system and a number of typhoid cases followed. It is assumed that he imagined that a higher pressure would be the only result of his work. This system of ours had operated successfully for some thirty years but it was not proof against deliberate tampering with the fixtures.

Again referring to the opportunities for *looking over* your plant don't *overlook* your accounting department. Investigation finds surprisingly few water bureaus using up-to-date methods in their book-keeping. In our cities you will find generally that the gas and electric corporations are the only ones having as many customers on their books as has the water company, and yet a great many of these smaller business and manufacturing concerns find that it pays to put in loose leaf ledgers, adding machines, billing machines, and a still more wonderful one, the book-keeping or posting.

A good many of us, I am afraid, especially in the municipally owned plants, are going on in the same old way handed down to us

from former city administrations perhaps dating back some thirty or forty years. In Rochester up to a very short time ago we were copying our accounts in longhand into some seventy immense ledgers, each one weighing on an average 50 pounds. They were too bulky to put away at night, or even to be moved around much during business hours, so they lay all day on long desks taking up a great deal of room and of course much time was taken in going from one to the other to look up accounts. We now have all of our accounts, some 50,000 in number, in twenty-one or two volumes, weighing about 15 pounds each. These are loose leaf ledgers with ten or twelve accounts to a page, and the sheets are ruled to last a year. At the end of the year new sheets are inserted and the old ones removed from the binders and stored away for as long as may seem expedient.

The consumers' names, addresses, etc., are put on the sheets by the addressing machine in the same order as on the bills and the meter readers' cards. The date, the meter reading, the amount consumed and corresponding charge are entered by the book-keeping machine which quickly balances by page, street or ward, and the whole scheme is working out very well. The entire city is metered, which, I think, simplifies the book-keeping of a water bureau. When we started the campaign for universal metering we went at it in the directly opposite method to the generally accepted plan, i.e., of metering the worst places and the most flagrant wasters of water first. We took at the beginning the better class of homes with the most modern plumbing and the more intelligent class of occupants. In a great many of these places, the water bills under the meter rates were not materially changed from their old rates and oftentimes the bills were smaller. It was not long before we had a great many meters installed without any serious opposition on the part of the consumers, and the more we installed the greater argument we had for putting in still more and getting everyone on the same basis, so that when we finally got down to the tough customers—those careless property owners with poor plumbing and wornout fixtures, who were extravagantly wasting water, why they had not a leg of an argument to stand on in their opposition to meters, and the majority of citizens were against them.

Our own meter men did the installing of all the meters and I think did it much better, cheaper and faster than if we had had it done by contract, and we were able to keep better records of the

meter installations. With three teams of two men each and a wagon, we averaged about 18 meters set a day—our record was 30 in one day. Our consumption is between 90 and 100 gallons per capita per day. Besides the sprinkling of the streets, we do a great deal of washing and flushing of the paved streets and are very generous in our use of water for parks, public bath-houses, drinking fountains and the like. We are also perhaps rather lenient in our hydrant rules. It has finally seemed to the writer impractical to keep the operation and use of hydrants confined to the fire and water departments.

The street department with the sprinkling and cleaning of streets and flushing of sewers, and the public utilities corporations, such as the gas, electric and street railways companies, and the contractors on street improvements; all these have to use water in the streets more or less frequently, and the hydrant is the most available and often the only means of obtaining water. With the beginning of spring, therefore, for such purposes, we attach to the nozzles of a great many hydrants a 2½" substantial valve, and on request other smaller or so-called contractors' valves. Neither of these has to be removed in case of a fire and they do not hinder the operation of the hydrant by the firemen. We have a certain number of men constantly looking after the hydrants, and with this method manage to keep them in pretty good shape. In winter these men with additional ones are kept constantly on the go to prevent or report frozen hydrants.

Repeated surveys show comparatively little leakage from mains and services, and all leaks and breaks when disclosed are fixed as promptly as possible. Considering the good condition of our piping system and the care we use to keep it so, I have come to the conclusion that a 100 gallon per capita rate is a fairly low one for cities of about 300,000 population or larger.

To properly maintain the system of mains, valves, hydrants, services, etc., of a water system, it is very necessary to have a good repair force; one of sufficient size and always available. In addition to our regular day force we always have on hand at nights, Sundays and all other holidays, a certain number of men ready for emergency calls and the whereabouts of most of the others is known so that they can be gotten hold of by telephone, automobiles or the like, in case of necessity. The key to our repair shop was practically thrown away forty years ago and the door has never been locked since then as someone is always there ready for action.

Unfortunately no one has yet been able to foretell where or when a water main is going to break. In a general way the old experienced water works man will tell you that they always break on the busiest corner of the most congested street on the coldest day of the year, with the crowds standing around wondering why it takes so long to shut the water off. It always does seem a fearfully long time before the gates can be closed, but I think as yet there is no better or quicker way than the old familiar, laborious method of hand-operation: that is, no method which would not be so excessive in cost, that the expense would far outweight the advantage gained by speed or ease of operation.

The number of bursting mains per mile of pipe per year, is after all—in spite of appearances—not so alarming. We have averaged about 7 per year for the last four or five years for some 500 miles of pipe. All sizes break and the larger ones do not always cause the most damage; it depends on the locality principally. Also all kinds break. The last serious break was in a 12 in. wrought iron main, which was caught by the long drawn out severe weather of the past winter and the freezing cold, and bursting, it had to be repaired or replaced at several different points. Query: Has the question of insuring water mains against breaking, ever been broached?

The speed in shutting off mains can be greatly helped if gates are properly located and easily found and the men thoroughly familiar with their operation. The gates in the large mains should be especially known and kept in working order. Care should be taken that they are not needlessly obstructed by building operations or rendered inaccessible in any other way. I call to mind that once when a 36″ main let loose, one of the gate covers as well as the whole street was quickly covered with a foot or two of rushing water and another gate up the street a block or two, was surmounted by a large temporary election voting booth which was too heavy to move without jacks.

The water covered gate could not of course be helped, but the placing of the election booth where it was never should have been allowed. It was what might be called a political blunder, something the water works superintendent frequently runs up against; but the politics side of this story must be told at another time, for already these experiences have run away with their narrator beyond the time limits and have, I fear, wearied his hearers, for I realize the comments have been rambling and the tale disconnected.

PRESIDENTIAL ADDRESS[1]

By Carleton E. Davis[2]

Concerning certain features of our Association and its work I have developed pronounced and perhaps radical views during the past year. One of these features which has been uppermost in the past two weeks concerns the customary Presidential address at the Annual Convention. I find no mandate in the Constitution requiring that the troubles of that official shall culminate in a so-called address to his fellow-members when, as matter of fact, he may feel that he should ask advice rather than give it, and plead for indulgence for his own shortcomings rather than attempt to make suggestions for the guidance of others.

My first radical view, therefore, is that the presidential address be omitted as a formal part of future Conventions. I congratulate the Program Committee on having omitted at the business sessions of the present Convention official addresses of welcome, responses, and other formalities. Likewise I congratulate the Publication Committee on having shortened the period of the Convention to a minimum and taken steps to see that the entire time of the Convention proper is utilized in fruitful business endeavor.

I feel that the widest future opportunity for this Association lies in a change in the relationship between the Sections and the central organization. The Sections must be developed—those already in existence must be strengthened, new sections must be created—all with a view of a final amalgamation in one central organization of the entire water works interests of the country. This process, carried to a logical end, will not destroy our existing organization but perhaps change its character, opening up to it a much wider field of usefulness along somewhat different lines.

Judicial opinion, as expressed in recent decisions, leans to the conclusion that a beneficent trust is not illegal. We have the

[1] Presented at the Montreal Convention, June 22, 1920.

[2] Retiring President, American Water Works Association; Chief, Bureau of Water, Philadelphia, Pa.

opportunity to create a beneficent trust—one that will work to the advantage of the water works operators and result in benefits to all the communities served by our utilities. To meet the situations that are before us to-day and that will arise in the years to come, to prevent lost motion and eliminate unnecessary duplication, all the water works of the country should function for their common interests through a central body which will be representative of all localities and which should be organized to meet all conditions and needs.

The American Water Works Association when it adopted its name expressed the intention to serve as this central body. The question before us to-day is whether we propose to grasp the opportunity to act or whether we shall let the opportunity pass us by. Acceptance of the present as we find it and committing the future to our successors is the easiest course. I believe our Association will elect to follow the more difficult but more fruitful path.

The majority of our members are restrained in their activities by regulations imposed upon them by public service bodies or by the restrictions of legislative enactments or by the limitations of city laws and ordinances. Many of these restraints are irksome and are not accepted calmly or without chafing under the restraint. Along the field of organization work we have a relatively free hand to organize and develop. We have an opportunity to demonstrate that the water works engineer and operator is a capable organizer and when given the opportunity can conduct his own affairs with efficiency and at the same time with due regard for the interests and welfare of the public, with an eye to the future and an alertness for the present. Necessary protests against undue outside restraint will come with much greater force if supported by the weight of a compact, comprehensive, well-organized and conducted Association.

It is stated on good authority that the investment in water works represents the largest total in the entire field of comparable utilities. Water works officials know that their plants, of all utilities, supply one universally, indispensable commodity—vital for human existence and basic for all community life. We have common interests, and that we are grasping for methods of expressing this interest by working together for a common end is shown by the number of water works organizations. The next step forward is the affiliation of the several organizations to handle matters of mutual interest with the power and weight that come from large numbers, unity of purpose and harmony of ideas.

Organization is the dominant note heard throughout engineering and allied fields at the present time. The American Water Works Association through accredited representatives recently attended a National Council in Washington of organizations concerned with public utilities like our own, seeking to coöperate offensively and defensively in matters vitally affecting the interests, personal and official, of their members. Whatever may be the final outcome of such efforts, or whatever may be the path the council of affiliated societies may elect to follow, self-respect demands that the water works of the country shall be able to speak as a unit at such councils and with the weight attaching to full and not partial numbers.

There is need for joint action and coöperation and it is for our Association to decide whether it will be the leader among all water works interests, and whether it will so conduct itself and its affairs that other water works organizations will feel the call to join it without undue proselytizing or the need of exerting pressure. I speak of proselytizing as applying to associations and not individuals, for no one can question the importance of increasing the membership in our own body during the period of change which I hope we are about to see.

Considered dispassionately, in cold blood, what does the American Water Works Association as now conducted, offer to its members?

First, is the JOURNAL which in my opinion is serving more and more to develop the Association, bind its members together, and attract increasing membership.

Second, is the privilege of attending once a year a Convention which of necessity is geographically inaccessible to a large portion of its membership.

Third, is the opportunity of voting by letter ballot for officers.

The Association is likewise the medium for the production of various standard specifications and standards of practice and custom, a matter which I would like to refer to later.

All of these openings and results are admirable, but the scope is limited, and most of us must conclude that the Association as now conducted, is not the active help to water works men that it should be and that it can be. Membership in it does comparatively little to promote the standing of the individual member in the community, protect him in a personal way, or place the utility which he conducts on the high level in the public mind to which it is entitled. The recognized official machinery of the Association is too remote

from the individual member and functions in too cumbersome a manner to respond to his needs or to be in his mind a tangible asset for his personal help.

The Association naturally thinks of itself as a parent Organization and the development of the Sections may appear as the breaking up of the family tree and weakening of the main stem. I feel that the contrary will be the case, provided the central organization is willing to look upon itself not as a major factor but as a clearing house—a medium of communication; or to draw an ambitious analogy—the Congress at Washington in its relation to the several States. Service rendered must be the underlying principle of the organization which attempts to weld into one functional unity, the entire water works interest of the continent.

We are holding this Convention in Canada—the home of a large number of our active, corporate and associate members. I feel confident that the further development of the principle underlying the Sections will tend to stimulate Canadian interest in the same measure that I believe it will tend to awaken interest in other portions of the continent, irrespective of geographical location. Water Works problems are physical and material, social and political. The first two mentioned are common, irrespective of locality. The last two mentioned depend upon the human element, and human nature has common characteristics the world over. Water works men on this continent have underlying interests and will continue to coöperate in the future as they have in the past.

We do not realize our strength because we have never attempted to exert it as a unit. Organized society is helpless against attacks and demands of relatively small but effective, because organized, groups working together. The existing lack of cohesion among water works men places them relatively in the position of the present unorganized public, powerful but helpless because of its inability to work together. Sometimes among our own ranks we hear references to the so-called little fellow and the big fellow. Each one of our members has something to give and something to receive. The perpetuation of the false idea of big and little, great and small, is harmful because it is based on wrong premises, is unreal, and exists only in the imagination.

The public water supplies of the country are developing from year to year meeting the growth of the country. They present highly complex technical and engineering problems. The period of

outside regulation has been with us for some time past and is increasing. Local, state and national health organizations are charged with functions relating to water supplies which are sometimes arbitrary and absolute. Public service commissions are given or assume increasing authority from year to year. Water works operators must determine for themselves whether they propose to accept, without a voice in the matter, standards which may be imposed upon them from the outside, or whether they propose to set up certain standards for themselves which outside bodies must of necessity accept because of the authority behind the conclusions of the water works operator. If the latter is to be the case, the water works men must take the initiative; otherwise it will be taken from them. If the water works men do not lead, they must follow.

Standards of practice and procedure must be done through Committees and Committee work will become of increasing importance. The path of the Committee must be smoothed as much as possible. When the authority for Committee work can emanate from a central body, representative of all the water works of the country, when coöperation comes logically and as a matter of course and not under option of possible rival organizations as at present, the desired end will be in sight.

SUGGESTIONS FOR A STANDARD FORM OF CONTRACT[1]

By G. W. Buchholz[2]

On behalf of the Executive Board and the Committee on Contracts of the Associated General Contractors of America, the author desires to express their appreciation of the privilege which he has tonight, of expressing their viewpoints on Standard Forms for Contracts.

As you are well aware this is a subject of vital importance to all constructors, for it is the heart of their business.

The legal definition of a contract is a meeting of the minds, but unfortunately for contractors, particularly, and for the construction industry as a whole, the engineers and lawyers have slowly but surely wandered away from that conception and through lack of initiative and coöperation by the constructors, have reached the attitude where they say, "If you want this job, sign on the dotted line" and the contractor signs.

In 1913 a group of contractors in Louisville, Kentucky, prepared some documents setting forth the inequalities and inequities in the

[1] Read at the Montreal Convention, June 24, 1920. At the St. Louis convention, the subject was brought up in an address by A. P. Greensfelder (see Journal, December, 1918) and referred to a committee consisting of G. W. Fuller, Leonard Metcalf, and E. E. Wall. This committee reported at the Buffalo convention (see Journal, September, 1919) and some of the members of the Associated General Contractors of America considered that the subject should be further studied. The Publication Committee consulted a number of members of the American Water Works Association with wide experience in contract work and finally invited the author to present this paper which would embody the views of his organization.

[2] Secretary, Associated General Contractors of America, Washington. The paper is signed by that organization's Committee on Contracts, consisting of J. W. Cowper, of John W. Cowper Company, Buffalo, *Chairman;* Arthur Bent, of Bent Brothers, Los Angeles; Max L. Cunningham, of the Municipal Improvements Company, Oklahoma City; Avery Brundage, Chicago; V. T. Goggin, of Fred T. Ley & Company, Inc., Springfield; L. S. Oakes, of Winston Brothers Company, Minneapolis; James W. Rollins, of Holbrook, Cabot & Rollins Corporation, New York; and E. Stanley Holland, of the Bates & Rogers Construction Company, Chicago.

building contracts then used by architects. From that small beginning, by persistent effort and the ultimate recognition and coöperation of the architects, the building contractors of the country were able to obtain a form of contract which is now known as the Standard Documents of the American Institute of Architects. These documents are, on the whole, very fair, reasonable and equitable for all concerned and evidence in a substantial manner a revival of the proper conception of a contract, viz., a meeting of the minds.

The Associated General Contractors of America are now in a position to initiate similar action tending to revise and equalize engineering contracts of all kinds and the author takes great pleasure in grasping this opportunity to state publicly his association's appreciation of the fairminded, cordial offers of interest and coöperation it has received from the leading engineers, engineering societies, public officials and public bodies of the country.

An analysis of income tax returns shows that contracting is the most hazardous industry in the country. They show that the amount of loss for every dollar of profit made by construction corporations is eight times as great as it is in manufacturing, agriculture, or personal service corporations; five times as great as in transportation and public utility corporations; over three times as great as in mining and quarrying; and nearly twice as great as in financing corporations.

The reasons for this condition are due to the unusual number of doubtful elements in construction with which a contractor must work. It is unnecessary to enumerate them. They are all too familiar. It is sufficient that the more such uncertainties can be eliminated, the lower will be the costs of construction to the owner; the more satisfactory will be the relations of the architect, the engineer and the contractor and the more will contracts be awarded on a basis of skill, integrity and responsibility. Never was this more true or action on it more needed than at the present time.

From the very nature of construction many of these uncertainties can never be eliminated. They must either be assumed by the contractor or the owner. Some which are now assumed by the contractor ought by right to be assumed by the owner. Other uncertainties may be eliminated entirely to the advantage of both parties. The improvement of present contract provisions is one of the simplest solutions for this problem.

With these thoughts in mind the Committee on Contracts of the Associated General Contractors of America during the past year has made a study of 113 different forms of contracts of various types. The committee is not prepared at this time to submit a complete form of proposal and contract embodying every provision which should be included in a good contract. It has, however, formulated the following basic provisions which it hereby submits for consideration and discussion. If consistent with your activities the committee recommends a joint conference of the proper committees of both associations on this subject and hopes your committee will be appointed at this annual meeting.

1. Action on bids. Bids should be submitted with the provision that they must be acted upon within a fixed time.

2. Freight rate changes. Bids should be submitted on the basis of existing freight rates and the contract should provide that in case a change in rates should occur between the time bids are received and the date fixed for the completion of the contract, the contract price should be increased or decreased accordingly.

3. Wage scale changes. Bids should be accompanied by a schedule of existing wage rates and the contract should provide that prices shall be increased or decreased in accordance with any change in such rates before the date fixed for the completion of the contract.

4. Material price changes. Bids should be submitted on the basis of existing prices for materials f. o. b. the producer's plant or distributor's yard, and the contract should provide that the contract price shall be increased or decreased in accordance with any change in such price that takes place within the time allowed the contractor to purchase and fabricate his materials.

5. Monthly estimates. Monthly estimates should include materials delivered and suitably stored as well as materials incorporated in the work.

6. Partial payments. Certificates should be prepared and delivered to the contractor between the first and tenth day of each month, showing the proportionate part of the contract price earned during the preceding month. These certificates should be paid by the owner by the tenth day of the month. Interest on deferred payments should be paid the contractor at the prevailing rate.

7. Contractor's right to stop work. Under the following conditions the contractor should have the right to stop work or terminate the contract upon three days' written notice to the owner and the

engineer, and recover from the owner payment for all work executed and any loss sustained upon any plant or material and reasonable profit and damages:

(a) If the work should be stopped under an order of any court, or other public authority, for a period of three months, through no act or fault of the contractor or any one employed by him;

(b) If the architect or engineer should fail to issue the monthly certificate for payment in accordance with the terms of contract;

(c) If the owner should fail to pay to the contractor, within seven days of its maturity and presentation, any sum certified by the architect or engineer or awarded by arbitration;

(d) If the owner does not permit the contractor to proceed with construction within a reasonable time after signing the contract.

8. Retained percentage. The retained percentage should be based on 100 per cent of the work performed and should never exceed 10 per cent. When the amount retained reaches a total sum which shall be mutually agreed upon by the owner and the contractor, no further reduction from payments should be made.

9. Surety bond. Where a surety bond is given, it should be reduced at agreed intervals so as to cover thereafter only that portion of work then uncompleted.

10. Penalty clauses. Wherever any provision is incorporated in the contract for a penalty against the contractor (including liquidated damages) there should also be inserted a provision for a bonus of like amount.

11. Acts of God or public enemy. The contractor should not be held liable for results arising from the acts of God or a public enemy.

12. Time allowed for completion of work. The time allowed for the completion of the work should be based on "Weather working days" instead of on elapsed time, and, if necessary, allowance should be made for time spent in performing unproductive work made necessary by floods or other natural causes beyond the control of the contractor.

13. Inspection. Where practicable, materials should be inspected at the source so that costly delay may not result from the rejection, at the site of the work, of materials furnished in good faith by the contractor.

14. Force account work. Payment for force account work should be made on the basis of the total actual costs of the work, including the actual labor and material costs, rental on equipment, liability

insurance and direct or field overhead, plus a reasonable profit to be agreed upon.

15. Change in quantities. In case the actual quantities of any item in a unit price contract are less than the estimated quantities by more than a certain fixed percentage, the unit price paid the contractor for that item should be increased by an amount to be agreed upon. Similarly a decrease in the unit prices should be made in case the quantities are increased over the estimate by more than a certain fixed percentage.

16. Arbitration. In no case should the engineer or architect be made the final judge as to the interpretation of the drawings and specifications or the performance of the contract. All decisions and interpretations should be subject to prompt arbitration at the choice of either party to the dispute.

Extras. In going over existing forms of water works contracts prepared by municipalities and consulting engineers selected at random, there was found under "Extra Work" a contract provision that unless "the contractor will file in writing all claims for extra work done within one month before the 15th day of the following month, the failure to file such claims within such time shall be deemed a waiver thereof and admission that no such claims exist."

Why should a contractor's oversight or even neglect to file a claim within a certain specified two weeks prohibit him from recovering what is justly due him (if he can so prove) even after the termination of the contract? Our laws include a statute of limitations usually extending two years. Why should a contractor be asked to sign an agreement which limits him to two weeks?

Here is another clause which is quite commonly used and which causes the wise and wary contractor to add another $100 or $1,000 to his proposal for uncalculable requirements in the contract: "Additional detailed plans will be furnished by the Engineer where necessary to show the construction of such details as are not fully shown upon the original plans, and the furnishing of such additional plans and the construction of the work there under shall not entitle the contractor to extra compensation." If the engineer when calling for bids cannot indicate clearly just what he desires to build, how in the world is the contractor to know what is wanted? He doesn't know and if he is a fool he "takes a chance" and adds nothing for such contingencies; if he is wise he increases his bid in proportion to the unknown quantity or detail and the public pays the bill.

In the same contract form just quoted appears another clause which unloads upon the contractor uncertainties and contingencies entirely beyond his control, yet he is expected to estimate the cost thereof and include them in his bid.

Protection of property, lights, etc. "The Contractor shall at his own expense shore-up, protect, and make good, as may be necessary all buildings, walls, fences, tracks or other property injured or liable to be injured during the progress of the work. The Contractor shall protect from injury all pipes, lamp-posts, and other fixtures that may be met with in carrying on the work, and in case any of these become damaged, they shall be immediately repaired by the contractor, or the parties having the control of same may make such repairs, and in this case, the expense thereof shall be deducted from any sum due or which may become due the Contractor."

Is it any wonder that proposals by contractors on the same piece of work sometimes vary 100 per cent, when they are forced to prejudge the cost of such uncertain and uncontrollable factors?

Subsurface Uncertainties. Still another clause which has a similar effect upon the amount of the item for contingencies in the contractor's proposal is that very common one making him responsible for all subsurface conditions and the refusal of the engineer to stand behind his own borings and surveys, as for instance, this one from a well known water supply contract.

(4) Responsibility for engineers' investigations of underground conditions. The contractor agrees that he has satisfied himself by his own investigation and research regarding all conditions affecting the work to be done . . . . he shall make no claim against the state because any of the estimates, tests or representations of any kind affecting the work made by any officer or agent of the state which may prove to be in any respect erroneous.

That clause has cost the public hundreds of thousands of dollars and put reputable contractors into bankruptcy. Yet why should the contractor be made responsible for conditions over which he had absolutely no control and cannot possibly foresee the cost of, unless he is a seer.

In closing the author would reiterate the recommendation and hope that the American Water Works Association will appoint a committee, if one does not already exist, to confer with the committee of the Associated General Contractors of America in considering these matters, with the abiding hope that some day we may obtain a Standard Form of Contract for Water Works of all kinds which will be absolutely fair, just and equitable for both contracting parties.

DIFFICULTIES IN BUILDING THE LOUISVILLE PUMPING STATION[1]

By James B. Wilson[2]

In the design of any subaqueous foundation where rock is not to be found at a reasonable depth, the controlling feature of any design is usually determined by the character of the soil which is to be penetrated, and the kind of material upon which the structure is to be finally founded. In many of our western rivers rock is far distant and when so located is usually overlaid with a deposit of alluvial sand and gravel which in turn is covered with fine sand, clay, quicksand and mud in varying combinations and thicknesses. In the work under discussion, rock was so far below the surface of the river that founding upon it would have been impossible except by very difficult and expensive operation.

Test holes were drilled over the entire area involved in the work and the nature of the soil carefully tabulated. The soil brought up from all test holes was similar in character and ranged from loam on the surface to compact sand and gravel at a depth of 80 feet below the river bed.

The type of foundation was then fixed for the new structure and several tentative designs were made, namely; a steel sheet pile cofferdam with steel truss bracing, which contemplated open excavation into which would be driven a wood pile foundation capped with a thick mat of concrete tied together with reinforcing bars, and on this mat to place the building walls; a wooden pneumatic caisson; a combined wood and concrete open-dredging-well caisson; and an all-concrete open-dredging-well caisson.

The first of these methods, the wood or concrete pile foundation with concrete mat, was discarded on account of the deep open excavation and the difficulty of driving the great number of wood or concrete piles. It was estimated a foundation 85 feet square would

[1] Read at the Montreal Convention, June 22, 1920. Discussion is desired and should be sent to the Editor.

[2] Chief Engineer and Superintendent, Louisville Water Company, Louisville, Ky.

require something like 720 piles, having a bearing power of 25 tons each, and spaced about 3 feet centers or closer. In view of the close spacing of piles it was found that the full load of 25 tons per pile could not be realized.

Adding to this the extreme danger to the surrounding buildings of the open excavation in case of accidents such as "blow-ins" or sudden movements of the wet soil underneath the enclosing cofferdam, thereby undermining an adjoining station, the surrounding embankments and structures, coupled with the high and rapidly increasing cost of structural steel of which the cofferdam was to be constructed, the plan was finally discarded as too costly and dangerous.

The steel sheet pile cofferdam, as designed for the above service, weighed about 833 tons and was made up of 50-foot steel sheet piles and steel trussed bracing. At the prevailing prices of fabricated steel and assuming a fair salvage price, an expenditure of about $50,000 would have been entailed for a temporary structure, the utility of which was seriously questioned.

A design was sought which would present features that would both be economical and the construction of which could be kept under absolute control at all times.

Three possible schemes then presented themselves as fulfilling the above requirements. The first was the wood pneumatic caisson, which, in view of the experience with that one on which old station was built, was rejected as unworthy of much consideration on account of its being too flexible an engine foundation, its high cost, and the serious objection of its not being able to be made sufficiently water tight to fulfill the requirement of a dry pump well.

The second was the combined wood and concrete open dredging well caisson, which was rejected for some such reasons as given for the wood pneumatic caisson, although its control would be almost absolute.

The third scheme was the consideration of an open dredging well caisson built wholly of reinforced concrete with steel cutting edges. Such a design presented features that were the most attractive of any considered. All the materials of which it would be constructed could be easily obtained in the local markets. Its construction presented no more difficulties than an ordinary concrete structure of equal size. It could be constructed on the bed of the river within a cofferdam in the dry, directly over the location where it would be sunk.

The difficulties attending the building of such a foundation would be those experienced in the sinking of so large a mass of reinforced concrete to the great depth required, and in controlling the sinking so that the caisson would attain its final position perfectly level and without having shifted horizontally either in one direction or another.

The final design was a caisson 90 feet square, with a bay on the river side 22 feet by 51 feet in plan, and a total depth of 33 feet in outside vertical dimensions. Sixteen octagonal dredging wells were

FIG. 1. APPEARANCE OF THE CAISSON ON JULY 10, 1917

placed in the main body and 2 rectangular wells in the river bay. The dimensions just given were determined by the outside dimensions of the upper foundation walls or substructure, the height of the main pump pit and the final depth of the caisson. It was deemed proper to carry down the caisson to the same depth as the wood caisson under the present station, or even lower. The idea was also incorporated in the design, of being able to convert the caisson from an open dredging well to a pneumatic caisson, if this became necessary from any cause whatsoever. The encountering

of some kind of hidden obstruction that would prevent sinking the caisson to its final designed position would require its conversion to a pneumatic caisson so as to make the obstruction accessible and its removal certain. Therefore, the cutting edges in the interior were raised over 5 feet so as to make two working chambers, one under the large portion and one under the bay, and by sealing the dredging wells with thick diaphragms of reinforced concrete, these chambers could be made air tight. This contingency was con-

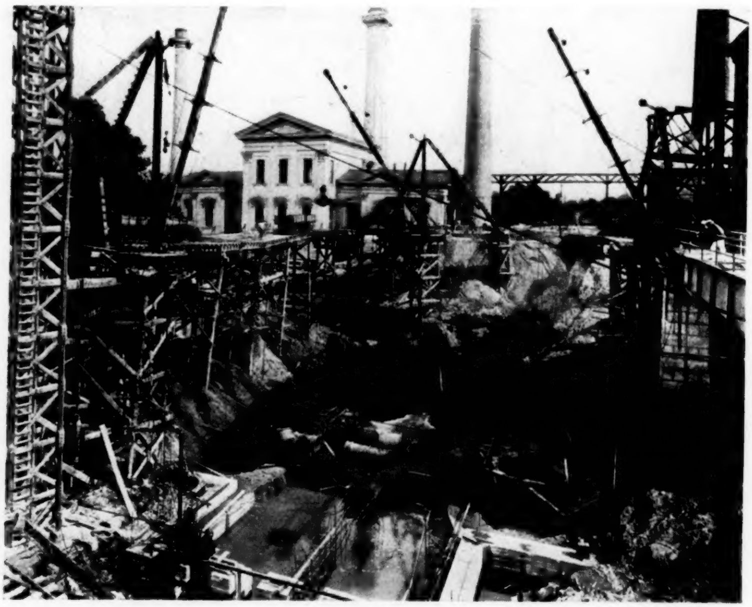

Fig. 2. Appearance of the Caisson on September 5, 1917

sidered rather remote in the light of the nature of the underlying soil.

The calculations in the design of the caisson were necessarily based on several assumptions, the nature of the soil to be penetrated, and past experience in sinking similar caissons. The first assumption made was that the caisson might be supported at any two diagonal corners, and the strength of the caisson was calculated to support its own weight neglecting skin friction and sinking weight of the caisson in its various stages of construction. As it was

assumed that only about half the caisson would be constructed before the first sinking operation would be commenced, and that a fair maximum skin friction for the kind of soil found was between 500 and 700 pounds per square foot of outer surface, weight enough would have to be provided to overcome the latter and allow the caisson to sink.

The caisson as designed contained 5858 cubic yards of concrete, 290,000 pounds of reinforcing steel, and 159,000 pounds of steel

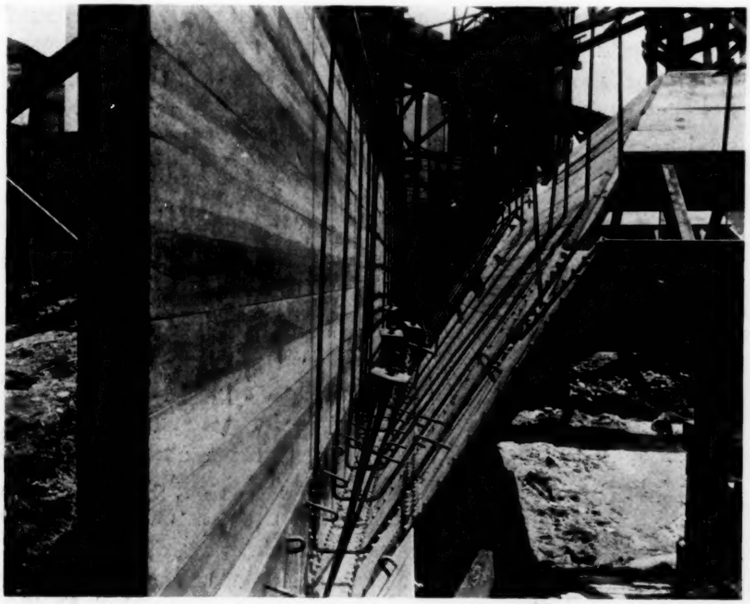

FIG. 3. SHOE AND REINFORCEMENT OF CUTTING EDGE OF THE CAISSON

cutting edges. The total sinking weight, therefore, was 23,195,000 pounds. This would overcome a maximum skin friction of 1739 pounds per square foot, provided there was no support under the cutting edges. As no such skin friction has ever been experienced in a purely sand and gravel soil without clay, it was concluded that the caisson would sink without any supplemental weight being employed. The actual skin friction was in excess of the above amount, due to the fact that the caisson refused to sink when free to move at the cutting edge.

The caisson was reinforced on all surfaces, both interior and exterior, against any surface cracks by 1-inch deformed bars placed horizontally and vertically. Horizontal bars were spaced on 18-inch centers, while vertical bars were spaced about 2 feet to 3 feet on centers.

Other reinforcing, 1-inch deformed bars spaced 6 inches on centers, was placed in horizontal layers at $10\frac{1}{2}$ feet, 18 feet and 26 feet respectively from the bottom of the lowest cutting edges, running in all the walls between the dredging wells and in the outside walls. The office of these bars was to allow the walls to act as horizontal beams to make the caisson self-supporting during the sinking operations. Diagonal bars of the same size were also introduced between the wells to tie the structure together efficiently in any diagonal direction. The wells were made octagonal especially for this purpose and to keep them as large as possible.

Two combined suction and screen wells were incorporated in the design of the caisson. This requirement was caused by the floor of the suction well being $10\frac{1}{2}$ feet below the main pump pit floor, which is the same elevation as the top of the caisson. The suction and screen wells occupy the river bay and serve as the effluent termini of the intake pipes. These pipes, four in number, are 36 inches in diameter as they enter the suction wells. They are connected to four 36-inch double disc gate valves which are electrically operated and controlled from the screen room above. The four pipes as they leave the face of the suction well bay connect in pairs to two 48-inch intake pipes coming out of the intake tower. One of these 48-inch pipes was in existence before this work commenced, it being changed somewhat in direction to accommodate the new connections.

The next step in the design of the station was that of the foundation walls, surrounding or forming the pump pit and the suction and screen well shafts. The requirements to be fulfilled were several: First, as the river level in its fluctuations of 40 feet would surround the substructure walls, they would have to be devoid of openings up to or above the highest known water in order to obtain a dry pump pit; second, they would have to be designed to resist the external pressure due to the highest stage of water; third, they would have to be similar in appearance to the corresponding portion of the old station.

The first and third of these were easily satisfied, but the second requirement, that of making the new station similar in appearance to the old, made it necessary to design the walls either as retaining walls of the buttress type, or of the gravity type. In both forms of walls it was found that the amount of reinforcing steel was enormous, and the lower thickness of walls would project too far toward the center of the building and encroach on the space for the proper placing of the pumps. The design finally adopted was that of making the pump pit walls circular within and square without, or in other words, to consider the walls as horizontal circular arch rings, fixing the arch elements 3 feet wide and 1 foot thick vertically.

Reinforcement consisting of 1-inch deformed steel bars was placed both vertically and horizontally in the walls at both outer and inner surfaces and spaced 18 inches on centers. Its principal function was to tie the walls together efficiently to prevent any shrinkage or settlement cracks which might produce leaks, and to stiffen the arch ring.

The substructure walls around the screen and suction wells were designed of vertical beams and struts. A division wall 3 feet thick divides the space within into two suction and screen wells. The wall runs from the floor of these wells up to the main engine-room or screen room floor. In this wall at its lower end, and as close to the floor as possible, was placed a 48-inch double disc gate valve, which allows the intake water coming into the suction and screen wells to be distributed from the east to the west well or vice-versa. This allows either pump to draw water from either well. Also when this valve is closed and the two 36-inch valves in the intake pipes are closed, in either well, while the corresponding pump is not operating, that well can be emptied of water by a sump pump and the mud and accumulated rubbish which have passed the outer intake screens can be cleaned out.

Before the work of constructing the caisson was begun, an earthen cofferdam was built entirely around the site of the new station. It consisted of two walls of wood sheeting, supported by horizontal lines of 8 x 10 inch wales and wood piles driven 12 feet apart. The two walls were tied together with 1-inch round rods and the space between the walls was then filled with puddle material dredged from the river. The height of the cofferdam was such as to with-

stand a rise of 15 feet in the river without flooding the enclosure and stopping the work.

The contractors' equipment consisted of the usual river equipment, including dredge boats, pile drivers, derricks, pumps, etc., and a land equipment of locomotive and dump cars, locomotive crane, stiff leg derricks, concrete mixers, etc. There was nothing unusual in the equipment to warrant further mention.

The sinking of the caisson by the open dredging well method is probably the only feature of any importance in the work under discussion as no mass of this area has been attempted, to the author's knowledge, without the use of air. With the eighteen dredging wells available for excavation and three stiff-leg derricks in operation with clam shell buckets, the sinking of the caisson was fairly rapid. The bottom of the excavation in the center of the caisson was maintained about 12 feet below the lowest cutting edge and in this condition the caisson was supported wholly around its outer periphery. When the caisson was within 3 feet of its final position the excavation by buckets was discontinued and preparation made to jet it down, as the sinking could be controlled better and as the surface of the excavation in the interior of the caisson was so much deeper than the interior cutting edges, the removal of more material was useless. The jetting was done from the inside of all the outside dredging walls, passing the jet pipe under the outside cutting edge from one well to the next, around the whole periphery of the caisson. The total sinking of the caisson was 38.28 feet, which was accomplished in twenty-eight days, actual digging time. The average sinking, therefore, amounted to 1.35 feet per day of twenty-two hours. The maximum in any one day was 2.15 feet.

It was thought advisable to allow the caisson to rest at this elevation, as a slight settlement would take place in the next two or three days, while the dredging wells were backfilled to the underside of the interior cutting edges. This was done so that communication from one well to another would be cut off, while the concrete seal of 6 feet was being placed. No settlement however took place.

Concreting the dredging wells was done by means of a 20-inch pipe or tremie placed in the water in the well and resting on the bottom. When the pipe was filled it was raised about 3 feet, allowing the concrete to flow out and become deposited in place. In this way all the dredging wells were sealed, taking them in order around the caisson.

The 30 feet of water remaining in each well was then pumped out and the seal found to be fairly tight. Such leaks as had developed were stopped and all wells were then filled by the usual method of pouring and the caisson completed.

Considering the entire weight of the pump well and superimposed walls, and the weight of the two 30,000,000-gallon pumping engines, it is interesting to note that the total bearing pressure on the sand and gravel upon which the caisson is founded is 4700 pounds per square foot.

NOTES ON RECENT DEVELOPMENTS IN CONCRETE[1]

By Lieut. Col. H. C. Boyden[2]

In 1914 the Structural Materials Research Laboratory was established at Lewis Institute, Chicago, with Prof. Duff A. Abrams at its head. The establishment of this laboratory was made possible through the coöperation of the Portland Cement Association and the Lewis Institute. This laboratory is a striking example of coöperation between an engineering college and a manufacturing industry of international scope.

There are only two ideas governing the policy of this laboratory; the first is, that the real facts regarding concrete and its ingredients shall be found out, with a liberal policy regarding the time required and the expense involved; the second is, that whatever the conclusions may be, they shall be given to the engineering profession for the improvement of the art of making concrete.

These investigations are still being carried on, but many points of vital importance have been established. As an example, the established data warrant the use of considerably higher unit stresses than those in common use today, with a consequent reduction in section.

Conclusions have also been arrived at that will enable us to obtain excellent results with aggregates heretofore condemned and to greatly increase the ability of concrete to resist wear.

These conclusions and many others, are all based on tests running into the thousands and covering long periods of time. The laboratory is equipped for and is making close to 50,000 tests a year, so that there is no lack of facilities for carrying out investigations in the most thorough manner.

The study of concrete may be conveniently divided into three phases:

1. The study of the characteristics of the ingredients.

[1] Presented by title at the Montreal Convention. Discussion is requested and should be sent to the Editor.

[2] With Portland Cement Association, 111 West Washington Street, Chicago.

2. The study of the effect of making various combinations of these ingredients.

3. The study of the effect of the various manipulations of the ingredients in making and curing the concrete.

It has been the custom to speak of concrete as having three ingredients, cement, fine aggregate and coarse aggregate. The laboratory studies have shown the desirability of classifying the ingredients as cement, aggregate and water, or if it is still desired to maintain the purely arbitrary division of the aggregate into fine and coarse, to add the fourth ingredient, water.

Although cement is one of the most important ingredients of concrete, it requires probably the least discussion, as all the standard brands of Portland cement on the market today conform to generally accepted specifications and the laboratory investigations have brought out no essential changes in these specifications.

As stated above, the aggregate has always been divided into two parts, sand and crushed stone or pebbles. The line of division, purely an arbitrary one, has been the quarter-inch screen; the portion passing through this screen has been classified as fine aggregate or sand, and that portion retained on the screen being called the coarse aggregate. There is no particular advantage gained by this division, but it would be much better to consider the aggregate as a whole, with a proper gradation of the various sizes from the largest to the smallest. It is not intended by this, however, to recommend the use of bank run or crusher run aggregate, as under no conditions should they be used without separating the sizes and recombining them in the proper proportions.

However, until such time as this method of considering the aggregate shall have become of general practice, it may be considered as divided into two parts by the $\frac{1}{4}$ inch or No. 4 screen, and the author will so discuss it.

Fine aggregate. It has been customary to specify that the fine aggregate shall be clean, sharp and not too fine. It would be better to use the word hard rather than sharp, as sharp sands give lower results than rounded or smooth sands. This is no doubt due to the larger amount of water required to obtain a workable mix when the sand is sharp or angular.

The laboratory studies have brought out two important facts regarding sands. One of these is the great importance of being sure that the material is clean, not only in appearance but in fact.

Very often sand which appears to the eye to be clean, contains enough humus or vegetable matter to reduce the strength of the concrete very considerably.

As an illustration a clean sand gave a compressive strength at 28 days of 1900 pounds. This same sand with one-tenth of one per cent of tannic acid added, gave a strength of only 1400 pounds; in other words, one-thousandth part of organic impurities in terms of the weight of the sand reduced the strength of the concrete nearly 25 per cent. In the investigation of the effect of organic impurities many natural sands were used, but as it was not feasible to secure sands containing a wide variation of organic impurities, tannic acid was used as a substitute for the purpose of making further tests. It was felt that the effect produced by such a material would probably be a measure of the effect produced by other organic impurities which might be present in natural sand.

How is it possible to detect these organic impurities if they cannot be seen by ordinary inspection? By using the colorimetric test for organic impurities devised at the laboratory. This test consists of digesting a representative sample of the sand in a dilute solution of sodium hydroxide (caustic soda = $NaOH$) and observing the resulting color of the liquid.

All that is needed is a 12-ounce prescription bottle and a little 3 per cent solution of caustic soda or sodium hydroxide, both obtainable at any drug store.

Put in about $4\frac{1}{2}$ ounces of the sand to be tested, fill up to the 7-ounce mark, after shaking, with the solution of caustic soda, let it stand for 24 hours and observe the liquid on top. If this liquid is clear or light straw colored use the sand; if it runs into the brown color and especially dark brown, reject the sand or wash it thoroughly before using.

The second fact is that fine sand behaves exactly the same as coarse sand with one exception. In order to produce a plastic workable mixture with fine sand it is necessary to use more water than with a coarse sand. It is the excess of water that reduces the strength of the concrete. In other words if it were possible to mix concrete with the same quantity of water regardless of the grading of the sand, and obtain a plastic mix in both cases, the same strength in the concrete would be obtained.

Coarse aggregate. When studying the characteristics of the coarse aggregate one conclusion has been brought out very sharply;

namely, that the hardness of the aggregate is a secondary consideration as compared with other factors in developing high crushing strength in concrete, and of less importance than ordinarily supposed in developing wearing qualities. This was very clearly shown in comparative tests made of burnt shale for use in building concrete ships. Samples made with this aggregate compared very favorably with those made up using a much harder aggregate. A stone must be very friable indeed if it is not strong enough, when properly combined in concrete, to more than maintain the load likely to be carried by the concrete.

The reason for high compressive results given where a light, soft aggregate is used is because the water content is reduced, owing to the porosity of the aggregate and not due to a higher compressive strength in the aggregate. The water content is again found to be the governing factor.

For road surfaces, however, another quality is needed in concrete, namely, wearing or abrasive quality, and to obtain this the stone must not be too soft. It is not advisable to use a stone with a French coefficient of less than 7 although pavements have given excellent results made with stone having a coefficient as low as 6.

It is not intended in calling attention to the above results to advise throwing down the bars and allowing the use of any and all stones, irrespective of their hardness or wearing qualities. It is desired, however, to show that many of the safeguards put into specifications in past years are not safeguards at all but that the effect of following them may be entirely lost through neglect to observe other factors of more vital importance. It is always advisable to use the best materials obtainable but there have been many cases, where the local and easily obtained material has been rejected, when it could have been used with excellent results, by following proper principles in proportioning and protecting the concrete.

Oftentimes better results would have been obtained than resulted from the use of imported materials and then neglecting the really important factors in making good concrete.

Water. The remaining ingredient of concrete, water, is in reality of equal importance with the cement in obtaining good concrete, and yet it is often the most carelessly used and most loosely specified of all the ingredients, generally not mentioned in specifications and frequently not even reported in test data.

The laboratory is now conducting tests of waters sent in from all parts of the country, but definite conclusions have not as yet been developed. It is safe to say, however, that waters which are strongly alkaline should not be used and, owing to the possibility that marsh waters may contain sufficient humus matter to seriously affect the strength of concrete, they should be looked upon with suspicion until tested in concrete and found satisfactory.

With regards to the temperature of the mixing water, tests have been made using water ranging in temperature from 32° to 212° F. It was found that the temperature of the mixing water had very little to do with the strength of the concrete at seven days to one year. The use of hot water is, however, a valuable aid in removing frost from the aggregate in cold weather, owing to its high specific heat, and may be used without danger of harming the concrete.

Proportioning. It is on studying the second phase of concrete making, that there has been brought out at the laboratory new, and in some ways radical, changes in the past and present practices of proportioning.

These investigations have brought out the following facts: That the present method of designing concrete mixtures by using arbitrary volumes is wrong, that there is one single proportion which will give the best results with a certain mixture of given fine and coarse aggregates, adding to or reducing the amount of cement is of value only as it affects the amount of water required to make a workable plastic mixture and above all, that the water-ratio is the most important element of a concrete mix. The water-ratio is the ratio of the volume of water to the volume of cement in the batch. If 1 cubic foot of water (7.5 gallons) is used for each sack of cement, the water-ratio is 1.00.

The use of more cement in a batch does not produce any beneficial effect except from the fact that a plastic, workable mix can be produced with a lower water-ratio. The reason that a rich mixture gives a higher strength than a leaner one is not that more cement is used, but because the concrete can be mixed with a water-ratio which is relatively lower for the richer mixture than for the lean one. If advantage is not taken of this possibility of reducing the water-ratio the additional cement in the richer mixture is wasted.

Fineness-modulus. In studying the results of the tests of many samples of various combinations of aggregates it was evident that there must be some relation between the size and grading of the

aggregates and the strength of the concrete. In trying to find this relation Professor Abrams struck upon what is called the "fineness modulus" of aggregates and when this was compared with the strengths of the concretes a direct relation was found to exist.

The fineness modulus is a very simple function of the sieve analysis of the aggregate used for any particular concrete. The sand and stone are analyzed with a set of Tyler standard sieves, each one of which has a clear opening double the width of the next smaller. The following sizes are used: 100 mesh, 48 mesh, 28 mesh, 14 mesh, 8 mesh, 4 mesh, $\frac{3}{8}$ inch, $\frac{3}{4}$ inch and $1\frac{1}{2}$ inch. The percentages (by volume or by weight) of the total aggregate coarser than each sieve are added together, the sum of these percentages is divided by 100 and the result is the fineness modulus. The fineness modulus of any combination of the fine and coarse aggregates may be found in exactly the same manner.

It is not possible to go into the details of the use of this factor for the design of concrete mixtures in a talk of this length but they have been published in *Engineering News-Record* on April 17, 1919, and a careful study will enable one to use it successfully.

It is not claimed that this method of designing concrete mixtures is the only one that will give the desired results, but the laboratory tests prove beyond a doubt that there is a direct relation between the compressive strength of concrete and the factor called the "fineness modulus." Accepting this as a fact, it is possible to design a concrete mixture that will give a certain desired compressive strength from almost any combination of aggregates.

Abrams' tables of proportions. In order to make this more easily available to the engineers of the country Professor Abrams has worked out a table containing 135 proportions with different combinations of aggregates, which if used with materials acceptable as to quality, will give a concrete with a compressive strength at 28 days of approximately 3000 pounds per square inch. All the tests for the determination of the factors in this table were made of a concrete of a workable plasticity, formed into cylinders 6 by 12 inches and tested at the end of 28 days.

In conformity with present practice the aggregate is divided in the table into fine and coarse, and covers combinations of five classes of fine aggregates with twenty-seven classes of coarse aggregates.

In order to determine what class a known aggregate shall be placed in, the following rules should be followed; if a fine aggregate at least 15 per cent of the total shall be retained on the next smaller sized sieve, if a coarse aggregate at least 10 per cent shall be retained in the same manner.

This table shows a considerable reduction in the amount of cement required as compared with previously published tables especially when combined with the larger sizes of aggregates. As an illustration, the quantities used today for a 1: 2: 3 mix, with sand up to No. 4 and stone from No. 4 to 1½ inch are 1.74 barrels cement, 0.52 cubic yard sand and 0.77 cubic yard of stone, while Professor Abrams' table calls for 1.61 barrels cement, 0.47 cubic yard sand and 0.72 cubic yard stone.

These figures are the exact quantities required for the making of one cubic yard of concrete and if used will effect a very material saving in the cost of the thousands of miles of concrete roads, pavements and other concrete structures to be built in the years to come.

An allowance for waste, varying for each ingredient and also according to the particular method employed in handling the work, should be added to the quantities given in the table. Professor Abrams is now preparing tables similar to the one already published, for concrete with a compressive strength of 2000 and 2500 pounds per square inch. As soon as these tables are completed they will be published in the technical press.

Water content. It is upon studying the water content that the most radical change from previous ideas on the design of concrete mixtures are found. It has been established that there is a direct connection between the amount of mixing water used and the strength of the concrete, and there is probably no other one factor which has so great an effect upon the strength as the water content.

It has been found that the less water used down to a certain point, the stronger will be the concrete, but this does not mean that one should go too far in reducing the amount of water, nor is it practicable in actual construction to reduce it to a point that would give the maximum strength shown in laboratory tests. There is another factor that must be taken into account in construction and that is the workability of the mix. In general terms, the lowest amount of water should be used that will give a workable mix.

The strength falls off very quickly with the addition of a small amount of water; so much so that in a one-bag batch the addition of one pint of water more than is necessary to give a workable mix produces the same loss in strength as leaving out 2 or 3 pounds of cement. This does not mean that it is possible to use a very lean mix with a small amount of water and obtain as strong a concrete as a rich mix with the same amount of water. This is not true, because it will require a higher water-ratio to produce a workable mix with the lean mixture, thereby causing a loss in strength.

The proper consistency for concrete will vary according to the use to be made of it. If the concrete is used for roads a drier consistency is practicable than for building walls containing reinforcing bars. The use of mechanical tamping and finishing machines in concrete road construction has made it possible to use the drier consistency economically, but any method which reduces the water content, such as the use of the light roller, will produce similar results.

The very wet sloppy mixtures that are being used in building construction may seem economical from the contractors' point of view but they are certainly extremely wasteful from the designers' and owners' point of view, as from 50 to 60 per cent of the possible strength of the concrete is being thrown away.

It may not be possible to reduce the amount of the water to the ratio necessary to give the maximum strength, but it certainly can be cut down below the amount commonly used, and the additional strength thus gained taken advantage of in the design of concrete structures. The designing engineer figures on a compressive strength of 650 pounds per square inch and expects to get a factor of safety of three or at least two, but does not get it with the sloppy mixture often used. By cutting down the water to the proper ratio, a factor of safety of five or six can be secured, and the allowable unit stress raised.

It is not possible to give the exact amount of water required for any particular mixture of aggregates to give the greatest strength in the concrete, owing to the impossibility of determining what amount will produce a workable mix and also due to the varying moisture content of the aggregate. A few approximate quantities for different proportions of well graded aggregates up to 1½ inch in size, may be given to form a basis for trial of the particular mixture at hand. A 1:2:4 mixture will require from 6 to 6½ gallons of water

per sack of cement, a 1:2:3 mix, $5\frac{3}{4}$ to $6\frac{1}{4}$ gallons and a 1:$1\frac{1}{2}$:3 mix, $5\frac{1}{2}$ to 6 gallons.

Slump test. In order to have a simple method for determining the proper consistency in the field the slump test has been devised.

When this was first devised a metal cylinder 6 inches in diameter and 12 inches high was used, but now a frustum of a cone 4 inches at the top and 8 inches at the bottom and 12 inches high has been adopted as a standard. This change was made owing to the tendency of the cylinder to lift portions of the concrete when it is being taken off and the dropping of this concrete causes variable results. The cone once loosened does not again come in contact with the concrete and the results are much more uniform. This cone is filled with the concrete to be tested, which is carefully worked with a metal rod while it is being placed, the form is immediately lifted off, and the settlement or slump measured. The proper slump for a mixture to be used for a concrete road surface is from $\frac{1}{2}$ to 1 inch; for mass work, from 1 to $1\frac{1}{2}$ inch and for concrete to be used in building walls with reinforcing bars $1\frac{1}{2}$ to 2 inches.

Manipulation of ingredients. In considering the final step, the manipulation of the ingredients during the making of the concrete, careful studies have been made of each operation. Included in this phase are the curing or protecting of the concrete during the early hardening period, as this is one of the vital operations in the making of good concrete.

The time of mixing is a matter of importance in obtaining good concrete and one that vitally affects the output of the mixer and consequently the cost. Exhaustive tests made on concrete mixed in a batch mixer from 15 seconds to 10 minutes, show a rapid increase in strength for the first minute, after which the increase is less pronounced as the time of mixing increases.

This shows the necessity of mixing the concrete at least 60 seconds and not 20 to 40 seconds only, as is often done in road and street construction. There is no question as to the advisability of using a batch meter on the mixer, provided one can be found that cannot be tampered with, in order to avoid controversy over the time of mixing and to insure a full minute mix. When a mixer is manufactured that will not permit discharge until a certain number of revolutions has been made at a certain speed this problem will have been solved.

The revolutions per minute of the mixer within the limits of 12 to 25 revolutions per minute have but very little effect on the

strength of the concrete, so that a sufficiently wide variation for different machines is permitted. In making tests of the effect of revolutions per minute on concrete the total time was one minute in all cases, and all materials, including water, were placed in the drum before the time interval was counted.

The effect of pressure on concrete immediately after moulding is found to be due to the amount of water squeezed out, making a consequent reduction of the water ratio. Tests were made on concrete of the same proportions, by applying pressure from zero to 500 pounds per square inch. The water expelled was carefully collected and measured. It was found the strength increased quite materially with the higher pressures and this increased strength was almost directly proportional to the amount of water squeezed out. It is not surprising to find, then, that the duration of the pressure had no effect whatever on the strength of the concrete. Whether pressure was applied for a few minutes or for several hours the effect produced was exactly the same.

It is undoubtedly the squeezing out of the water and consequent reduction of water-ratio that produces the excellent results when the roller method of finishing concrete roads is used.

The time that can be allowed between the time of mixing and the time of placing has not as yet been made the subject of extensive tests at the laboratory. This knowledge is of value when considered in conjunction with central mixing plants, which are used with success in many places. The lapsed time is undoubtedly governed to a certain extent by the kind of cement used, by the temperature of the ingredients and by the temperature of the mixed concrete. In Illinois a limit of 40 minutes lapsed time is allowed but it is generally believed that the economical haul for the job will be the governing factor rather than the fixing of a time limit.

It is possible that some of our present ideas regarding this factor may be changed by the results of such a series of tests, but until such a time it would not be advisable to allow re-tempering of concrete that has been too long in transit, as the addition of water will no doubt result in a reduction in strength.

Protection. The proper protection of concrete during the early hardening period is a detail of construction that is only too often overlooked and many times only indifferently carried out.

The effect of proper curing conditions upon the ability of the concrete to withstand abrasion has been very strongly brought out by numerous tests in the laboratory. There is probably no method

of handling concrete that so affects its wearing ability, as that of providing proper protection while curing or hardening.

It is true that any and all of the factors that tend to produce strength in concrete also tend to increase its wearing qualities, nevertheless all of the tests show that, other factors being the same, the concrete which is properly protected will show much less wear than that which has been allowed to dry out too quickly. As an illustration of this the strength of a concrete of 1.25 consistency was about 1700 pounds per square inch when it was allowed to dry out in the air unprotected and exactly the same concrete stored in damp sand for 21 days gave a strength of about 4000 pounds per square inch and a correspondingly less wear under the rattler test.

One of the principal causes of the poor wearing resistance that we sometimes find in concrete floors is due to the practice of allowing them to dry out without proper protection during the hardening period.

Concrete floors under roof should be covered and kept moist just as outside roads and pavements are protected. Why throw away one half the life of a concrete floor by failing to observe this rule and holding back from using them for so short a period?

The essential requirements for proper hardening are warmth and the presence of moisture, especially the latter. The tests show a less increase in wearing resistance and strength after 21 days have elapsed and a constant rate of increase during this period. In deciding on the length of time that a pavement, or other structure, shall be kept covered and moist, it is simply a matter of deciding how much of the potential strength and wear resisting qualities it is desirable to throw away and reduce the 21 day period by that amount.

There are several methods of protecting concrete pavements during this period, the most effective of which is the ponding method and where the grades and other conditions will permit this method to be used, it will give the most lasting results. The protection of concrete structures other than pavements is very often either neglected altogether or at best only half carried out. Many times the leaving on of the forms is considered to be sufficient protection in itself, but this is not so.

The forms and all exposed surfaces should be kept thoroughly wet, or at least very moist continuously for not less than 14 days and whenever possible for 21 days or more.

COST PLUS CONTRACTS FOR WATER WORKS CONSTRUCTION[1]

By George W. Fuller[2]

Prior to the Great War, the "cost plus" form of payment on contracts in the water works field was limited to a relatively few large projects built as a whole under this type of contract for private corporations and to numerous small unexpected features of enterprises executed under municipal contracts where "extra work" clauses were attached to either lump sum (bulk) or unit price contracts.

During the war a large amount of emergency government work which had to be performed in the shortest possible time gave great impetus to the "cost plus" form of contract or what the British call "prime cost plus profit" type of contract.

The unstable condition of the market for labor and materials now found in many places causes this form of handling construction work to come up repeatedly for discussion. Such discussion results from the necessity for finding expedients to meet present emergencies which, while not comparable with those of the war period, are nevertheless present during this reconstruction period to an extent which perhaps is not generally recognized.

At this time when contractors are sorely puzzled to know how to bid or tender on construction materials on which quotations are made by dealers only on the basis of changes in price contingent on the actual date of future deliveries and when labor is uncertain in quantity and of reduced and somewhat uncertain efficiency as to output of work per hour, it is obviously necessary to look conditions squarely in the face. Add to this the difficulties in transportation of construction materials and the loss incident to the contractor having a substantial payroll for labor when materials to work with are lacking and it is readily seen that this is a time for considering

[1] Presented at the Montreal Convention, June 24, 1920. Discussion is desired and should be sent to the Editor.

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fundamental principles in handling construction work to an extent that would not be of interest under normal conditions.

The author believes that water works construction which reasonably can be deferred should not be contracted for at present. In the case of many water works projects the existing works can with propriety be patched and overhauled in a manner similar to that adopted by the individual who under present stress makes use of old and patched clothes. There are some water works betterments which cannot be postponed owing to the fact that this expedient has been resorted to for so long that further postponing means positive disaster along various lines associated with a water famine.

Much needed work is now impossible of accomplishment due to the inability of labor agencies to supply men, of manufacturers to supply material and of transportation facilities to deliver goods. Competition between owners will only increase the already serious aspect of obtaining material for construction work which it is important to have done in the near future.

Some water works construction must go forward. With conditions as they are at present the contractor, if he bids on a lump sum or unit price basis, is bound to name a price which in his opinion will protect him from loss and if possible assure a reasonable return on his capital investment and for the work of himself and his organization. Under these circumstances it is important to discuss briefly the cost plus form of contract with a view to seeing if the burden of uncertainty in some respects cannot be shifted from the contractor to the owner to the advantage of all concerned. In fact, if construction work is to go forward there are some projects where such steps seem imperative.

Legality of cost plus contract. Before pointing out briefly the advantages and disadvantages of the cost plus form of contract, it is well to recall that there is serious doubt as to the legal right of municipalities, in some localities at least, to enter into such contracts. State and provincial laws and municipal charters usually call for the letting of contracts for public works by open competitive bidding for all work in excess of certain specified amounts. Also such laws provide, almost without exception, that contracts shall not be made in sums in excess of appropriations duly authorized and in some cases above the estimates of the engineer of the governing body.

As regards the first restriction the intent is obviously to take advantage of competition in making sure of proper market prices.

The second restriction is a check on total estimated costs by actual bids for the work. In the absence of preliminary bidding, work might be started when appropriations are insufficient for completion, or opportunities might be lost for reducing costs by subsequent lettings through correction of misunderstandings as to construction procedures.

Stabilization through such restrictions should not be abandoned unless there is a great emergency.

Cost plus contracts. There have been and always will be certain construction problems in which some means of payment in the form of actual cost plus a percentage or lump sum is legitimate and desirable, if not necessary. This is true even with lump sum contracts in which certain features may be indefinite or subject to changes regarding the character of the work. Cost plus provisions for extra work, or supplementary contracts based on such arrangements or estimates, are also pertinent where sufficient investigation previous to a letting is undesirable or too expensive in time or money for the advantage gained, such as extensive street openings to locate existing structures, and also in cases where the owner desires to retain complete control of the work regardless of the cost.

It is claimed that under the cost plus method a contractor has little incentive to keep down the cost of the work. This is frequently true of the cost plus percentage but need not be true of the cost plus lump sum type. In any case it must be remembered that a contractor who will deliberately be inefficient on a cost plus project is equally sure to attempt improper or inadequate construction on lump sum or unit price agreements.

There have been a great many variations of the cost plus contract applied to construction work but the more important are:

1. Actual proved cost with labor and material furnished without restriction by the contractor—plus a fixed percentage or lump sum to represent profit, supervision, financing, use of tools and plant, or any or all of these.

2. Actual proved cost of labor furnished by the contractor and with materials furnished by the owner, with a fixed percentage or lump sum as above.

3. Actual proved total cost for specified work plus a percentage for specified or unexpected extra or unforeseen work in connection with lump sum or unit price contracts.

4. Actual proved total cost to the contractor plus a sliding scale fee and upset maximum fee.

5. Actual proved total cost to the contractor plus a fixed plant charge and fixed construction fee.

Advantages claimed for cost plus contracts. 1. The work may be started at any time and is not dependent on the prior completion of the plans.

2. The owner may radically increase or decrease the quantities during construction, with lump sum fees subject to review.

3. The owner may change the kind of construction during the progress of the work.

4. The contractor will not try to skimp the job as is often done after finding himself losing under lump sum or unit price contracts.

5. There is less need of having accurate preliminary estimates. In unit price contracts the preliminary estimate is often so approximate as to cause the contractor to gamble on some of the items, with resulting disappointment to the owner in the ultimate cost of the work. Such procedures sometimes cause an unfair result either to owner or contractor or both.

6. Unit price or lump sum bids require a set of specification definitions which are not always clear and are sometimes deficient and sometimes overlapping. Such ambiguities may lead to arguments and variations in cost which result in claims and suits for extra work.

7. Cost plus contracts may save the owner the money which the contractor usually adds to his lump sum or unit price bid to cover the complete but actually infrequent enforcement of material tests which cause him delay and expense, or necessitate the carrying of a large stock, entailing interest charges, storage space, rehandling, etc.

8. Cost plus contracts do away with the substantial sums usually added in lump sum or unit price contracts to cover the following uncertainties: (a) Weather; (b) foundations; (c) changes and shortages in labor market; (d) changes and shortages in material market; (e) delayed deliveries of materials.

9. For cost plus work it is the contract and not the specifications which is the crux of the matter from both the owner's and contractor's viewpoint. The writing of the contract is more simple than the writing of the specifications.

10. Cost plus contracts tend to promote coöperation between the owner and the contractor.

Disadvantages claimed for cost plus contracts. 1. There is no way of determining the approximate cost in advance and this upsets budgets where definite appropriations have been made or are required.

2. Competition—the key to efficiency—is killed.

3. Greater opportunity is offered for favoritism on the part of the owner's representative.

4. Where the same contractor has several jobs, the lump sum and unit price contracts will get the good workmen and the cost plus contract will get the drones and misfits.

5. A large general contractor often takes a job on the cost plus basis and sublets it to several smaller contractors on a lump sum or unit price basis. In making the sub-contractors complete the work on the latter basis the general contractor often treats them unfairly while he himself may be receiving a substantial profit for doing little or nothing.

6. Should work be started before plans are completed, many errors may have to be straightened out in cases where competent engineers would avoid them if given an opportunity to get out a complete set of plans and accurate estimates of quantities.

7. Engineers are tempted to be less thorough in their work when they know that there will be no comeback at them on account of extras resulting from their failure to have plans and specifications complete when needed. With cost plus contracts it may simply be a matter of correcting an error or supplying a deficiency when discovered, but the cost is there just the same although it may not appear as an "extra."

8. Engineers or other representatives of the owner must do an immense amount of accounting and clerical work in checking payrolls, material bills, etc., and expend much time and energy in expediting the delivery of materials.

9. A premium may be put on extravagances and waste by giving unscrupulous contractors and engineers a chance to take advantage of the owner.

10. The contractor may procrastinate in securing, if not refuse to secure, promptly adequate tools and equipment as to type and number.

11. A combination of the above disadvantages, although no one by itself may be sufficiently pronounced to permit the owner successfully to obtain relief, may cause grief for the owner, unless protected

by a maximum fee to the contractor, and by the assured adequacy of the latter's organization and equipment.

Pre-War Status of Cost Plus Percentage Contracts. Much large construction work was and is done by railroads and other corporations in this way but this is done usually because these corporations unlike municipalities are legally able to select competent contractors with efficient organizations and equipment to work under the immediate direction of a skilled and alert staff of the owner who for the most part purchases supplies and materials direct. The Grand Central Terminal in New York is a case in point. Much of the work was first awarded to a contractor on a unit price contract but it was completed under a cost plus agreement.

There can be no question that some work can be as economically and efficiently done under the cost plus basis as in other ways. But in the water works field such work appears to form the exception rather than the rule under peace time conditions.

Army contracts for construction work. One of the best examples of cost plus percentage agreements is found in the United States Army construction work during 1917–'18. Here the contract was on what is known as the "cost-plus-a-sliding-scale-fee" with a maximum upset fee.

Overhead expenses and interest costs reduced the actual profit to most cantonment contractors to less than one and one-half per cent. In this case the specified cost of the work included all payments of whatever nature with the exception of overhead costs of the contractor including financing expenses.

The union scale of wages prevailing in the locality of the work under consideration as of June 1, 1917, was agreed upon by the Secretary of War and the President of the American Federation of Labor as the scale for the camp. Rules in reference to overtime, and for the regulation of hours of work, were agreed upon in advance between the constructing quartermaster and the contractor, in accordance with conditions prevailing in the district where the work was done.

The question of whether or not the Government got a fair return on its investment depended largely on the adequacy of the auditing and checking system which the constructing quartermaster carried out on the job.

There is no room for doubt as to the question of the United States military establishment having acted wisely in adopting the cost

plus form of contract for its emergency work. Speed was of vital importance. Abnormally high speed always means abnormally high cost. But in this war emergency abnormal construction costs were of no significance as compared with the saving in blood and general war cost which resulted from such construction speed.

In this connection it is well to recall that the government through priority arrangements had the benefits arising from the commandeering not only of materials and of labor but also of transportation. Such benefits do not attach to peace time work in any field.

General considerations regarding contract work. On contract work engineers should not be compelled to do the work of both the engineer and the contractor because contractors should be more capable than owners or their representatives to handle advantageously and economically the details of construction requirements. Contracts should therefore be drawn in such a manner as not to limit the work and responsibility of the contractor to the furnishing of labor and to the execution of details under the absolute direction of the owner and his engineer. Responsibility for good construction and final excellency of the work should rest with the contractor.

To insure satisfactory results from cost plus lump sum agreements the contracts should provide for the reimbursement of the contractor for all amounts actually spent by him, such expenditures being limited in the case of materials to their normal market value and for labor to price schedules of local labor unions. The lump sum fee allowance should include the services of the works superintendent and hand tools and such small equipment as would obviously be required for the work. For plant equipment such as machine tools, excavators, cable ways, etc., a per diem rental under stipulated conditions should be fixed or bids should be received. To prevent disputes the plans and specifications should be as carefully prepared as for other types of contract.

Good examples of cost plus *lump sum* agreements are found in the United States Housing Bureau and other government contracts which provide for a rental price for plant equipment and a contractor's fee for services, on all of which bids were taken from a list of acceptable contractors.

The plant equipment fee covered the rental of all machinery, scrapers, scaffolding and tools, and all material which did not enter into permanent work. It did not include, however, lumber for temporary forms, concrete centering or the bracing and supporting

of these forms or light staging or scaffolding on the exterior of structures.

The contractor from his fee was obliged to pay all costs of transporting, loading and unloading the plant equipment, and all upkeep and maintenance charges, fuel, oil, office overhead and general superintendency. A bonus was allowed the contractor, in addition to the lump sum fee which he bid, in the amount of one-fourth of any saving he was able to make as compared with his accepted estimated cost of the total work.

Under this agreement the contractor financed the entire construction work and was reimbursed by the owner for all such cost. A very strict system of inspection and accounting was maintained by the government.

Profit sharing methods have considerable merit over the straight cost plus forms in that they give the contractor some incentive to keep the cost of the work down. A method which has been used with considerable success in Canada is described by R. O. L. French. By its terms the contractor receives 20 per cent of the estimated cost and rebates to the owner 10 per cent of the actual cost.

Contract adjusted to varying labor prices. Morris R. Sherrerd, Chief Engineer of the North Jersey District Water Supply Commission, in a recent contract for the construction of the Wanaque Dam, provided for an adjustment of certain labor costs after the year 1920, provided such costs are 10 per cent above or below normal 1920 prices. This places the burden of changes in material costs on the contractor, but causes the owners to share with the contractor unusual changes in labor costs. The advantage of this type of contract as to labor over cost plus agreements lies in the fact that the contractor is compelled to exercise the same careful supervision and that there is the same necessity for economical construction methods as is required on lump sum and unit price agreements, but he is not obliged to shoulder all responsibility for unexpected price changes.

In the Wanaque Dam contract, labor is a controlling item, but on ordinary construction work, particularly on comparatively small jobs, and where the values of labor and material are more nearly equal, there would be less advantage in it. To be more generally applicable the adjustment in prices should if possible include material as well as labor, and furthermore the length of time between successive adjustments should also be made to con-

form to the size of the contract and to the probable duration of construction.

The adjustment of prices is more difficult in the case of materials than for labor, because of the greater number of materials and also because of the variety of materials which might satisfy any particular specification. On this account the furnishing of the principal materials to the general contractor through separate contracts made by the owner may be advantageous in that the risk would be more widely distributed and the adjustment of prices made somewhat more simple and definite.

At a time when transportation facilities are abnormally inadequate the assumption by the owner of the responsibility of furnishing materials cannot eliminate wholly the troubles arising from the enforced intermittent use of the contractor's laborers.

Summary and conclusions. 1. Pre-war construction contracts were for the most part, and rightly so, agreements on a lump sum or a unit price basis. Cost plus contracts were used only on certain large work done for private corporations or as a part of other types of agreements.

2. During the war the United States Government construction and much other work was done on a cost plus basis. Where proposals on a lump sum or unit price basis were obtained, the prices were intended to be sufficiently high to insure against loss due to constantly changing prices and the scarcity of labor and material.

3. Since the war, the procedure has been somewhat unsettled, with an effort to do away with some of the disadvantages of cost plus form and to combine so far as possible the good qualities of both types of contracts.

4. The unit price contract under normal stable market and transportation conditions is the most satisfactory. The lump sum contract is principally advantageous in that the final cost is definitely known at the outset.

5. Cost plus contracts, with proper provision for accounting and supervision, may be satisfactory where conditions are not definitely known and in the case of private corporations where well qualified contractors may be selected to work under adequate supervision. Under war conditions cost plus contracts were necessary and even now have many advantages.

6. Construction work for private corporations may be successfully carried out with proper safeguards under any of the discussed

forms of contract. For general construction work under municipal control the nearer a contract approaches the well-established lump sum or unit price contracts, if indeed any departure from such contracts is legal, the more satisfactory will be the results secured.

7. Until such time, however, as the material and labor markets are better stabilized, contracts should in fairness place the burden of uncertainty on the owner and not on the contractor. This may be done as follows:

(a) For much municipal work a form of contract may be adopted along the lines proposed by Mr. Sherrerd, and modified as suggested as to labor and material adjustments at proper intervals.

(b) For municipal or other work contracts may provide for the furnishing by the Contractor of such labor and materials as are reasonably stable, with adjustment for changes in the labor market, and with materials of unstable price furnished by the owner through separate contracts.

8. Construction work not absolutely necessary should be deferred, and materials and labor should be diverted so far as possible to work which is absolutely necessary.

9. So far as possible necessary improvements should be made by repairing or enlarging present works, and new works should be confined for the present to immediate needs.

THE BASIC PRINCIPLES USED IN THE DESIGNS FOR THE NEW WATER SUPPLY WORKS OF WINNIPEG, MANITOBA[1]

By James H. Fuertes[2]

Winnipeg is situated about 65 miles north of the international boundary, and about 35 miles south of the southern end of Lake Winnipeg, at the junction of the Red River, which rises in Minnesota and follows a northerly course, and the Assiniboine River joining it from the west, the combined waters continuing north to an outlet into Lake Winnipeg. The city, prior to its organization as a town, was merely a small settlement or trading post, called Fort Garry, at the junction of these two rivers. In 1874 the population of Winnipeg is given as 1869; by 1890 it had jumped to 23,000; by 1910 to 132,720. By 1913 the population of the entire Greater Winnipeg Water District was 215,000; the present population is estimated to be 250,000.

History. The new water works of Winnipeg, forming the subject of this paper, were built to provide a satisfactorily large supply of soft water. Before the construction of these new works the city depended upon a ground water supply of very hard water, unfit, without softening, for manufacturing or commercial uses, and too limited in quantity even for domestic use. Under such conditions the establishment at Winnipeg of manufacturing or industrial works requiring a plentiful supply of suitable water was out of the question and a great handicap to the proper development of her otherwise excellent resources as a trade center. The question of changing the water supply had been agitated annually for a number of years, but the inevitable expense involved in going a great distance to secure a new supply necessitated the postponement of active steps in that direction a number of times.

[1] Read at the Montreal Convention, June 23, 1920. A supplementary paper read at the same time by Wm. G. Chase, describing the construction of the Winnipeg System, will be printed later. Discussion of this paper is requested and should be sent to the Editor.

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Prior to 1880 the municipal water supply of Winnipeg was obtained from wells scattered about the town, and was distributed to the consumers from tanks and barrels on carts in the summer, or sleds in the winter. In 1882 the Winnipeg Water Works Company, operating under an exclusive twenty year franchise, built a water works plant on the Assiniboine River on Armstrong Point, establishing a pumping station and mechanical filter plant, of the pressure type, the filtered water being pumped directly into the city's street mains. This system was purchased by the city in April, 1899, for a little over a quarter of a million dollars, but was abandoned in 1905 following the introduction of a ground water supply.

The quality of the water purveyed from the Assiniboine plant was not satisfactory, nor was the ground water supply, both being very hard. Agitation in favor of a new and better supply was kept up more or less continuously from that time until the commencement of the building of the present new supply.

In March, 1883, Dr. Agnew in a letter to the *Free Press* directed attention to the Lake of the Woods as a source to which Winnipeg must ultimately look for her water supply, and again elaborated upon this source in February, 1884, in an address before the Manitoba Historical and Scientific Society. In February, 1895, Walter Moberley, C.E., made a report to the City Council advocating a new supply from the Winnipeg River. Nothing was done with either suggestion, however, and in October, 1896, Col. H. N. Ruttan, city engineer, reported on the relative merits of a supply from the Assiniboine River and a supply from artesian wells. The well supply was developed quite extensively; the water, however, while agreeable to the taste and excellent as to appearance, was too hard for ordinary municipal purposes and in September, 1897, Dr. Rudolph Hering was asked to report upon the possibility of softening this water and on the merits of other available sources of supply. Dr. Hering examined and reported on a ground water supply from Poplar Springs, a pumped supply from the Assiniboine River, a pumped supply from Winnipeg River, and on the extension and softening of the artesian well supply, recommending that the softened ground water supply be adopted and further developed.

About 1905, the city, having grown very rapidly in population and the ground water supply having been much depleted, so far as local sources were concerned, another examination was made and reported on to Councils in 1907 by Messrs. Fuertes, Lea, Schwitzer

and Whipple, who investigated the possibilities of extending the ground water supply, securing a supply from the Red River and the Assiniboine River, as well as supplies from the Winnipeg River, and from the Lake of the Woods. This report favored developing a supply from Winnipeg River as being less expensive than a supply from the Lake of the Woods, while being equally satisfactory, after proper treatment. Following this report another investigation was made by Prof. C. S. Slichter for Public Utility Commissioner H. A. Robson, favoring a supply from the Lake of the Woods or Shoal Lake; and in the following year, 1913, the question of the desirability and availability of the Shoal Lake supply was again referred to a commission of engineers composed of Dr. Rudolph Hering, Frederick P. Stearns and James H. Fuertes, who after considering the possibilities of a gravity supply to Winnipeg from the Lake of the Woods, through a concrete aqueduct, concluded that works could be built of much larger capacity than had heretofore been considered for a sum which it would not be too difficult for the district to provide. The report was adopted by the officials of the Greater Winnipeg Water District on September 6, 1913, by the City Council of Winnipeg on September 8, 1913, and voted on affirmatively by popular vote on October 1, 1913. On October 20 five field parties were on the ground making the preliminary surveys for the alignment.

THE NEW SUPPLY

Organization. Since Shoal Lake waters are tributary to the waters of the Lake of the Woods, a portion of which crosses the international boundary into the United States, it was necessary to secure for the project the approval of the International Joint Commission having jurisdiction over boundary waters; and as the boundary line between the provinces of Manitoba and Ontario passed through Indian Bay, a tributary of Shoal Lake, it was necessary also to secure the consent of the Ontario Government to the taking of these waters.

The Greater Winnipeg Water District, consisting of the city of Winnipeg and six smaller neighboring municipalities, was organized and constituted by proclamation of the lieutenant governor of Manitoba, June 10, 1913. The Water District was authorized to go outside of Manitoba for water by the Dominion Parliament in Act 3–4, George V, Chapter 208, and on October 2, 1913, an order in Council of the Province of Ontario was passed permitting the use of water

for the Winnipeg Water District from Shoal Lake up to a limit of 100,000,000 gallons per day. The International Joint Commission approved the application January 15, 1914.

All difficulties in regard to the securing of the water having been settled, surveys and investigations were immediately started and by the end of February, 1914, the location was completed, there having been involved the taking of 380 square miles of topography, 362 miles of transit lines, 1,317 miles of levels, 95 miles of precise levels, 11,544 feet of soundings in Indian Bay, and 3,897 feet of test borings along the line of the aqueduct. The work of clearing the right-of-way was started in March, 1914, and finished in about three months, the standard width of the right-of-way being 300 feet; greater widths were secured where necessary. The first actual construction work was the erection of the telephone line, 91 miles long, which was begun May 5, 1914, and cost about $32,500. In October, 1914, contracts were awarded for the whole aqueduct from Deacon to Shoal Lake. The first water was turned through the completed aqueduct and discharged into the McPhillips street reservoir in Winnipeg on March 26, 1919. The general features of the system are outlined in figure 1.

General description of country traversed by the aqueduct. It was found, from studies and inspection, that the line proposed in the Hering, Stearns, Fuertes report, which was based partly upon surveys made specially for that report, and partly upon interpolated profiles based upon the profiles of the Canadian Pacific and Grand Trunk Railways, was a practicable line, but that a more intimate knowledge of the country on either side of this line gave promise of securing a more economical aqueduct than that following the line proposed. Throughout the whole length of the line the country is very flat and was largely covered with swamps, timber and underbrush. Only in a few places was it possible to see off to any distance from the line, and actual elevations had to be taken along the section lines, or wherever clearings had been cut out, for a distance, sometimes, as much as ten miles either side of the line, in order to avoid running into impracticable country on the one hand, and in order to be able to pick out a better alignment on the other hand.

For about 20 miles eastward from Winnipeg, the plains, formed by the deposition of clays from suspension in the sea water of the ancient glacial Lake Agassiz, have a slope of but 1 to 3 feet to the mile. To the eastward of this plain, the country rises somewhat more rapidly for a few miles. Large deposits of gravel occur in

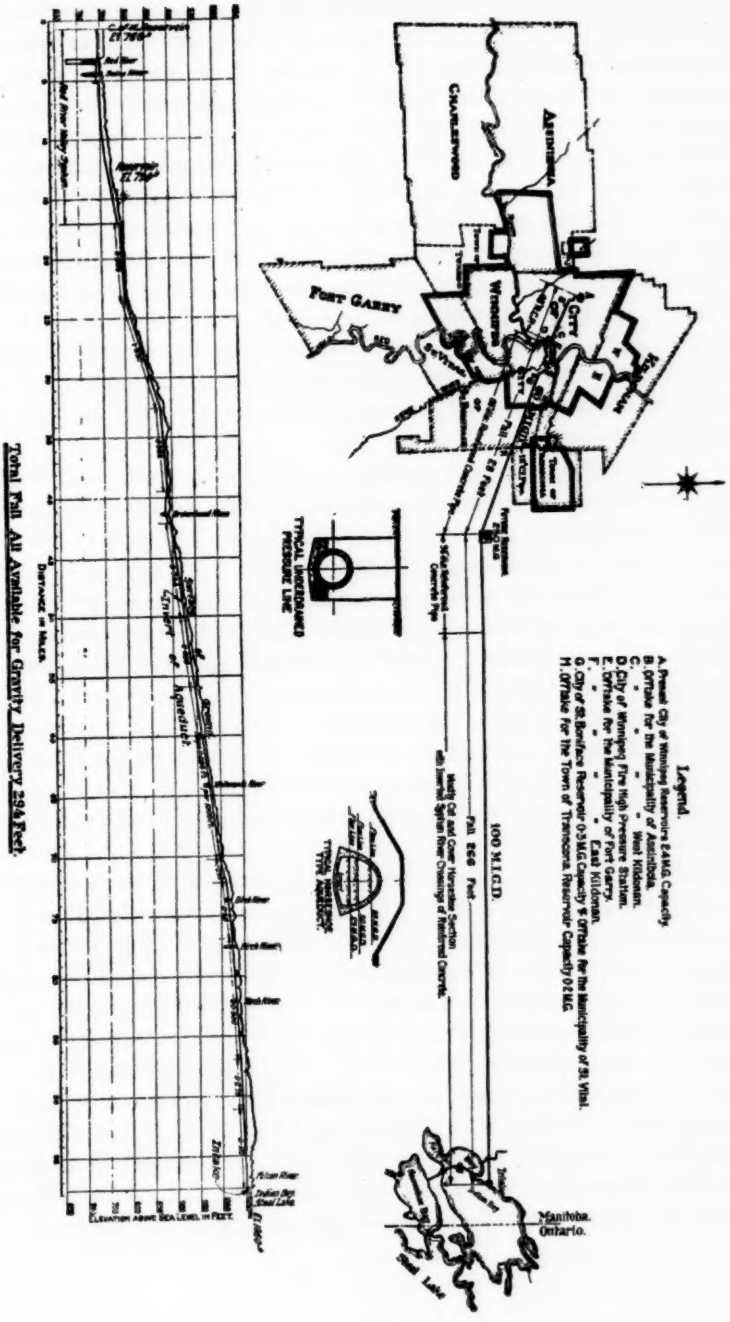

FIG. 1. GENERAL FEATURES OF THE GREATER WINNIPEG WATER SUPPLY WORKS

ridges along what were, in ancient times, progressive shore lines of this inland sea as the continental ice-cap receded. Where the shore line followed along the contour of the ground, the glacial deposits were washed out leaving the sand and gravel comparatively clean and free from clay. Going still further east and, in fact, located more or less along the whole length of the aqueduct, the soil is a mixture of clay, sand, and gravel, sometimes stratified, sometimes mixed, on top of which has grown up successive generations of vegetation forming what are locally termed "muskegs." The sub-soil throughout the whole distance is practically water-tight, and consequently the run-off of rain and melting snow, which amount together to only about 22 inches from the watershed in a year, is retarded so greatly that but three open streams are crossed by the aqueduct in the nearly 100 miles between Shoal Lake and the Seine River, which joins Red River at Winnipeg.

Source of supply. The report of Hering, Stearns and Fuertes recommended securing the water from Indian Bay, that portion of the Lake of the Woods nearest to Winnipeg. Indian Bay is an arm of Shoal Lake, and Shoal Lake, which has a water area of 110 square miles, is connected with the main Lake of the Woods by means of a narrow channel called Ash Rapids. While the watershed of Shoal Lake, 360 square miles, is not large enough to maintain in very dry years a yield of 85,000,000 gallons a day, this yield can be had by drawing through it on the main body of water in the Lake of the Woods without appreciably affecting the water level of the main lake. The fluctuation in level between low and high water in Indian Bay or Shoal Lake lies between elevations 1059.6 and 1065.0 above sea level.

Indian Bay, from which the water is taken, is about 6 miles long and from 1 to 3 miles wide. Numbers of soundings lengthwise and crosswise of this lake show its depth to vary from 24 to 26 feet over the whole bottom, except in the immediate vicinity of the shores, which shoal up more or less rapidly. At the western end of Indian Bay a stream known as Falcon River enters and discharges the highly colored swamp waters lying to the northwest of Indian Bay. Just to the south of and parallel with Indian Bay lies Snowshoe Bay. A promontory varying from one-half a mile to a mile or more in width, and with a length of about six miles, separates Snowshoe Bay from Indian Bay. Both Snowshoe and Indian Bays connect openly with Shoal Lake at the eastern end of the promontory above mentioned.

Decolorization. The waters of all the bays, tributary to Lake of the Woods, are more or less highly colored, being fed by small streams having their origin in the swamps and muskegs bordering the main lake. The color of the waters of Shoal Lake and of the main Lake of the Woods, away from the influence of these small highly colored streams, and where time and the bleaching action of sunlight have had an opportunity to bring about decolorization, is not high enough to be objectionable for ordinary municipal uses. Indian Bay, when first examined, was decidedly discolored throughout its whole area, the greater part of the discoloration coming from Falcon River.

A study of the topography of the promontory lying between Snowshoe and Indian Bays disclosed a low area through which a canal could be cut at small expense, to connect the two lakes, which suggested the plan, subsequently adopted, of diverting the black waters of Snake Lake and Falcon River through this canal into Snowshoe Bay by the building of a dyke across the shallow water at the west end of Indian Bay. The cost of this dyke and canal, which secured an intake point from which low colored water could be had in Indian Bay, was about $147,000; to extend this aqueduct 5 miles further, to Shoal Lake, the only other alternative to secure satisfactory water, would have cost about $1,000,000. The results have been quite satisfactory, as may be seen from the statement that on June 7, 1915, the diversion dyke having been completed in 1914, the color of the water on the Falcon River side of the dyke across the end of Indian Bay was 107 while the color on the Indian Bay side of the dyke on the same day was 9 (platinum cobalt scale).

The natural color of the water in Indian Bay is lower in the winter than in the summer; under the ice, the color remains practically constant during the entire winter. During the winter of 1919–1920 the color at the intake remained at 12.

The bottom of Indian Bay is more or less covered with accumulations of leaves, shells of infusoria, dead leaves, grasses and other matters of organic and mineral origin, generally reduced by time and natural agencies to a stable or non-putrescible condition. In addition, the waters contain numbers of algae and living organisms of various kinds. The lake being comparatively shallow, the waters experience two distinct turnovers each year, due to the changes in temperature above and below the temperature of maximum density. Ice forms on the lake to a thickness of about 4 feet and the temperature of the water drops to about 34° at the bottom of the lake in

very long continued cold weather. When water, at the surface of the lake, on the approach of cold weather reaches a temperature of 39° it tends to sink to the bottom of the lake and displace the warmer and lighter water underneath. This continues until the water of the entire lake has reached a graded temperature such that the heaviest remains at the bottom and the lightest at the top. In the following Spring, when the surface water again warms up, the warmth being transmitted gradually to the lower strata, as soon as that at the surface reaches a temperature of 39° it sinks, as before, displacing the colder water beneath, and a gradual readjustment of the entire mass takes place.

There are thus two distinct periods when bottom water will be brought to the surface, and with each over-turning, of course, some of the matters which have settled to the bottom will be caught in the rising currents and brought up to the surface. This phenomenon is one which required watching for many reasons, one of which was discovered during the first year of operation of the new works. On the occasion referred to, such large quantities of grasses and weeds were brought up during the over-turn that the intake screens, before thought was given the matter by the local attendant, became clogged sufficiently to cause a head of several inches on the screens, with the consequent breaking out of the wire mesh, permitting dirty water and fish to pass into the aqueduct.

BASIS OF DESIGN FOR WORKS

Design of intake. The design of the lake intake was studied out with a view to preventing troubles that might interrupt the supply of water to the city. These troubles, it was anticipated, would be due largely to ice in the extremely cold winter weather. It was planned, therefore, to allow for an ice sheet 4 feet thick on the lake and to have a passage beneath this sufficiently wide and deep enough to admit the water into the intake with as little loss of head as possible and without drawing in spicules of ice forming on the under side of the ice sheet.

The general plan was to extend two rock ballasted earthen dykes, with side slopes of 1 on 3, in parallel lines out from the shore 200 feet to where the water was about 22 feet deep, at high water, the dykes being 180 feet apart on their center lines The space between these dykes was dredged out and the whole area covered with screened

gravel, bringing the finished surface up to elevation 1048, or about 13 feet below mean lake level. The front wall of the concrete gate house was carried down to a depth of 8.7 feet below average lake level, or 6.25 feet below low water, in order to cut off the entrance of cold air into the screen chamber through the intake, and in order to prevent the formation of ice in the intake structure. The screen chamber itself was simply an extension of the aqueduct with its bottom widened and its sides made vertical. This chamber was divided into two parts by a longitudinal partition and two sets of screens were placed within in such a way as to offer a large area to the moving water and permit of removal for cleaning. The whole screen house was covered with an earth embankment 4 feet deep for frost-proofing, so that ice would not form on the fine mesh screens and retard the passage of the water.

Air temperatures as low as 50° below zero are not infrequent at the intake and commonly temperatures of zero and lower prevail continuously from the beginning of December until the end of March. Frost, therefore, was a matter to be reckoned with in the design for these works. The general arrangement of the intake structure is shown in figure 2.

The screens had a total submerged area at high water of about 700 square feet, which, for a water consumption of 100,000,000 gallons[3] per day would correspond with 7 square feet per million gallons per day. The screens were of copper wire cloth having $\frac{3}{8}$-inch mesh backed by a screen with 1-inch meshes. After the accident of the breaking through of the screens last summer, an additional set of vertical steel racks with $1\frac{1}{2}$ inch spacing was installed just below the upper stop log slot; a second set of screens with six meshes to the inch was also installed in connection with the coarser screen to guard against a possible repetition of that unfortunate and unpleasant experience. The three barriers have been successful in excluding undesirable water-borne life.

The screens are lifted for cleaning by a chain hoist running on overhead tracks. Inside the screen house below the screens, there is a boat entrance to permit the dropping of a boat into the aqueduct for the purposes of inspection from the lake to the inverted siphon at the Venturi meter under Falcon River. Another boat entrance is placed just beyond Falcon River to permit of inspection from Fal-

[3] The imperial gallon, not the United States gallon, is the unit employed in Canada and referred to in this paper.

con River to Birch River, and again, at the beginning and end of each of the inverted siphons to the end of the arched section 5 miles west of Deacon.

Temperature of water flowing through aqueduct. As to temperatures of the water at Indian Bay, as compared with the temperatures in the city reservoir after passage through the 100 miles of aqueduct, it may be remarked that there is practically no difference at any season of the year. The temperature at the intake, as well as at the city reservoir, gradually decreases, from the time the ice begins to form on the lake in November until a minimum of 2°C. (35.5°F.) is reached in December at the reservoir, persisting until March; from the time of the break-up in the spring, generally in April, there is a gradual rise in temperature until a maximum of 22.5°C. (72.5°F.) was reached in July and August, being followed then by a gradual dropping until the freeze-up.

In the distribution system of the city the temperature of the water rises somewhat in winter and falls in summer. In the heart of the city, where the pipes are large and the quantities of water used are large, the temperatures do not vary more than a degree or two from those in the city reservoir; approaching the limits of the city there is a gradual rise in winter to a temperature of about 2.5°C. (38.5°F.) and in summer a gradual drop to a minimum of about 6°C. (43°F.).

The outstanding feature respecting temperatures is that throughout those cold winters, when the cold penetrates the ground sufficiently deep to cause thick hoar frost to collect on the arch of the aqueduct above the flowing water, enough heat is radiated into the water from the ground to a little more than balance that lost at the surface during winter, and the reverse during summer to enable this water to keep at a practically constant temperature while flowing about 100 miles.

This result bears out the predictions made in the report of 1913 that 4 feet of cover would be a sufficient protection to the aqueduct, as far as frost-proofing was concerned, although it was known that in some localities frost penetrated as deep as 9 feet in cold winters; in fact, in Winnipeg, small service mains freeze sometimes in June and July, at that depth.

Prior to the completion of the aqueduct, at the suggestion of General Ruttan, formerly city engineer of Winnipeg for many years, an experiment was made in a completed portion of the aqueduct to reproduce, as well as might be, the conditions to be expected of the

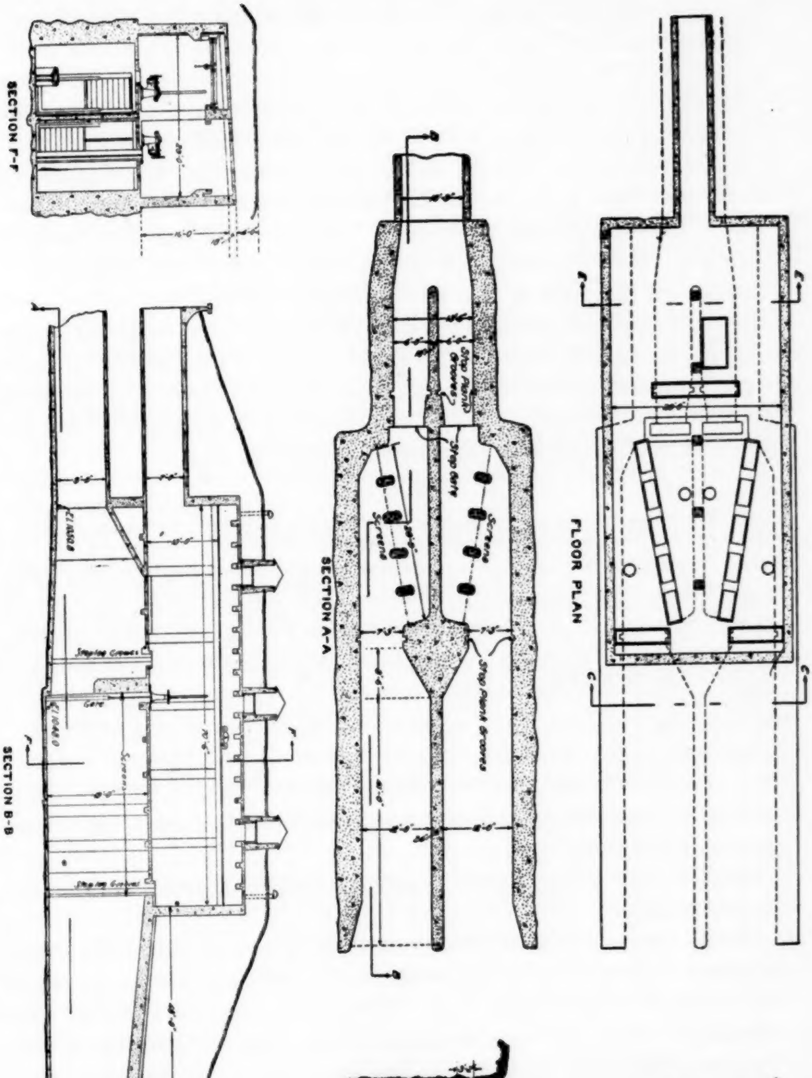

FIG. 2. THE INTAKE OF THE WINNIPEG AQUEDUCT AT INDIAN BAY

aqueduct when in operation. A section of aqueduct was chosen, running westward from the Birch River crossing, at which point water could be diverted into the auqeduct, and turned out again after passing through 3 miles of the aqueduct into a ditch at the west end of the section under test. As the aqueduct slope was too steep to control the flow by a dam at the lower end, three sand-bag dams were built in it, one at the lower end, and two others at intervals of about 4000 feet upstream. This aqueduct had the standard 4 feet of cover of sandy, loamy material. Water was regulated to enter from the river at the rate of 130,000 gallons per day, and at that rate required about a week to travel the three miles before escaping, being held back by the dams, to subject the water to aqueduct temperature for about the same time that would be required for water to flow from Lake of the Woods to Winnipeg at the minimum rate of operation, 25,000,000 gallons per day.

TABLE 1

Winter temperatures in test section of Winnipeg aqueduct, February 2, 1917

	OUTSIDE AIR	AIR IN AQUEDUCT	WATER IN RIVER	WATER IN AQUEDUCT	DEPTH OF WATER, AQUEDUCT
	degs. F.	*degs. F.*	*degs. F.*	*degs. F.*	*inches*
	−14		32		
Entrance to aqueduct		29		33	3½
First dam		36		36	40
Second dam		37		37	40
Last dam		37		37	56

The first set of temperatures was taken on February 2, 1917; they are recorded in table 1.

No ice nor frost appeared on roofs or walls of aqueduct nor on the water flowing.

From the last dam to the second, the depth of water decreased gradually from 56 to 3½ inches in about 3500 feet, and remained at 3½ inches for about 1000 feet. Between the second and first dam the depth decreased from 40 inches at the dam to 3½ inches in 2500 feet, and remained 3½ inches to the first dam, about 1500 feet.

Observations repeated in March gave practically the same results, although the outside air had risen to 32°F., and the long continued cold had penetrated the backfill over the aqueduct, causing a strip of hoar frost 2 to 6 feet wide to form to a depth of ¼ inch on the inside of the top of the aqueduct.

LINES AND GRADES FOR AQUEDUCT

Economical depth of cut. The establishment of the proper alignment for the aqueduct involved a study of costs balanced against grade lines. The preliminary profile prepared for the report of 1913 furnished the base for comparisons. The cost of an aqueduct section on any given grade varied with the position of the invert with respect to the ground level. Very shallow cuts, requiring large embankment quantities for covering the aqueduct, and very deep cuttings cost a great deal more per foot than a moderate average cut. Comparative estimates of cost, including the stripping, excavation, timber platform, concrete and backfilling showed that, for local conditions and methods of payment, an average cut of from 4 feet for the smallest to 5.5 feet for the largest section gave the minimum costs per foot of completed work, and that with a range of half a foot either side of the most economical depth there was practically no difference in cost.

After the preliminary line had been run through from Winnipeg to Shoal Lake it was found that there were three critical points through which this line had to pass, one was the west end of Snake Lake, near the Lake of the Woods, another was the crossing of Brokenhead River, where moving the line very far either side of the chosen crossing would greatly affect adversely the grade line or increase the length of the inverted syphon required to cross this valley or plain; and the last was the passing from the Brokenhead to the English River drainage basins, where there was but one low saddle to follow without swinging the line far away from the short route.

Foundation conditions. With these points fixed, the problem resolved itself into finding the most suitable alignment, for the least cost, to connect the points into a continuous line. Had the country been dry or the sub-soil the same throughout, the problem would have been much simpler, but, in places, although the natural ground level was at a favorable height, the sub-soil was so wet and soft as to be unsuitable to carry the aqueduct, requiring either, 1, a pile foundation; 2, the removal of the soft ground to a solid bottom and refilling of the trench with gravel hauled in by train and deposited in the trench in water kept standing high enough to entirely submerge the gravel until the surface was raised a few inches above the finished grade of the aqueduct (many miles of foundation had to be

prepared in this way); or 3, the depression of the aqueduct sufficiently below the hydraulic grade to secure a solid bottom and the reinforcement of the concrete work so as to stand the resulting internal pressure.

As finally located there were nine of these depressed sections, as shown in table 2.

Effect of slope on cost of aqueduct. The general problem as to alignment, therefore, was one of finding the line giving a grade averaging as close to the mean slope between termini as possible without diverging so far from the shortest practicable line as to have the extra length cost more than would be saved in concrete, excavation and

TABLE 2

Depressed sections on Winnipeg aqueduct

LOCATION	DIAMETER		LENGTH	MAXIMUM PRESSURE HEAD
	feet	*inches*	*feet*	*feet*
McPhillips St. Reservoir to Red River	4	0	11,400	*20 to 65
Red River to Deacon	5	6	49,900	20 to 50
Deacon, 658 + 60 to 900 + 30	8	0	24,140	10 to 30
Brokenhead River, 2080 + 00 to 2220 + 00	7	6	14,000	7 to 25
Whitemouth River 3383 + 77 to 3386 + 25	6	9½	248	26
Birch River, 3888 + 27.6 to 3890 + 17.5	7	3	190	20
1st Boggy River, 4098 + 80 to 4101 + 00	8	0	220	26
2nd Boggy River, 4332 + 95 to 4334 + 85	8	0	190	21
Falcon River, 5105 + 40 to 5110 + 232†	9	0	483	15 to 20
Total length			100,771	

* Tested to 40 pounds per square inch.

† Includes Venturi meter.

foundation costs by using a longer alignment on a flatter slope. Thus, with the 7-foot by 8-foot 3¾-inch aqueduct the line could be lengthened 14 per cent to avoid a cut 6 feet greater than the most economical depth of cut.

Except at the critical points spoken of, the entire country was so flat, so heavily covered with brush and timber, or so wet and boggy, that the eye was no guide in picking out the proper line. Levels had to be run on offsets from the preliminary lines, sometimes as far as 20 miles at right angles, along section lines that had been cut out, to get a comprehensive contour map, with contours at 1 or 2 foot intervals, to trace out thereon the best line to follow to secure

good grades, good lines and avoid difficulties in other places. In addition to this, soundings and wash borings had to be put down in soft places and in swampy areas to locate the hard subsoil, and determine whether to use a foundation fill, depressed section or pile foundation to get the aqueduct through, and determine the cost.

Hydraulic gradient at east end of aqueduct. In the establishment of the grade of the aqueduct there were some novel points. The first relates to the passage of the aqueduct from Indian Bay to the valley of Boggy River, through a deep cut 9 miles long. The height of the water in Indian Bay fixed the range of levels available to govern the entrance of the water into the aqueduct, and the depth of cutting, influenced greatly by the grade of the aqueduct, controlled also the size of the section and hence the cost. Naturally a very slight slope was adopted for the long 9-mile cut, at the end of which the grade steepened considerably, and this circumstance was utilized in the following way:

The grade adopted for the 9-mile cut was 0.11 foot per 1000, and the depth of flow on this slope to discharge 85,000,000 gallons daily, through the 10 foot 9 inch by 9 foot section of aqueduct, would be 7.25 feet. The slope of the next section was 0.279 foot per 1000 feet and the depth of flow, with the 8 foot 10$\frac{1}{4}$ inch by 7 foot 5$\frac{5}{8}$ inch section, 6.26 feet for 85,000,000 gallons daily discharge. By continuing the large section past the junction of the two grades and down on the steeper slope for 11,100 feet, then changing to the smaller section, the water surface at the junction of the flat and the steep slopes would drop to give a depth of flow of 4.98 feet instead of the 7.25 feet required to discharge the same quantity on the 0.11-foot grade, and the larger aqueduct would have the advantage of this additional slope without the necessity of dropping the aqueduct grade.

In other words, this expedient to secure discharging capacity made possible the placing of the invert grade through the deep 9-mile cut an average of about 10 inches higher than if the large section had ended and the smaller begun at the break of grade, and still discharge the required 85,000,000 gallons per day at low water in Indian Bay. Also, using the 10 foot 9 inch by 9 foot aqueduct had the further advantage that at high water in the lake the capacity of this large section would be about 120,000,000 gallons per day, so that at the very slight additional cost required for the larger section, this portion, the most expensive of all the aqueduct, would not

require duplication for a very long time in the future. These conditions are shown in figure 3.

Provision for second aqueduct on increasing supply in future. Should, in the future, the rate of consumption pass 85,000,000 gallons, it is contemplated to install a booster pumping station to raise the lake water the few feet required, when required, and run the aqueduct section under a slight head, from nothing to about 4 feet, for the first 5 miles of its length. Provision was made in the works as built, for the junction of a second aqueduct to the existing one at the end of this extended large section, so that advantage could be taken of these conditions when the demand for water exceeds the capacity of the existing aqueduct, without interfering with the supply of water to the District.

General description of aqueduct. From this point to the site of the proposed equalizing reservoir at Deacon, about 12 miles from Winnipeg, the problem of grades was simply one of adjusting the alignment to secure that line which could be built for the least cost, in other words, equating lengths and corresponding unit costs as determined from the resulting grades and foundation difficulties to give the least cost between termini. Usually, in locating gravity aqueducts, there is little choice in selecting a route, topographical difficulties controlling and limiting the location to one practicable line. In the case of the Winnipeg aqueduct, however, the country was so flat and its general slope so uniform, that, as far as slopes were concerned, an aqueduct could have been run through on an almost straight line between Winnipeg and Shoal Lake. Such an alignment would, however, have cost from $2,000,000 to $3,000,000 more than the one finally located, although several miles shorter, as it would have consisted of long stretches of nearly level grades, joined by short stretches of steep grades, and the resulting aqueduct would have been made up of a large percentage of large sections and a small percentage of small sizes of aqueduct.

From the intake westward, to within about 17 miles of Winnipeg, a distance of 80 miles, the slope of the ground is such that a gravity aqueduct, planned to run not quite full, was used. The section is horseshoe-shaped, as will be shown later, except where depressed below the hydraulic grade. These portions are circular and reinforced for internal pressures. The largest gravity flow section is that through the summit cut, above described, which is 10 feet 9 inches wide and 9 feet high on a slope of 0.58 foot per mile; the

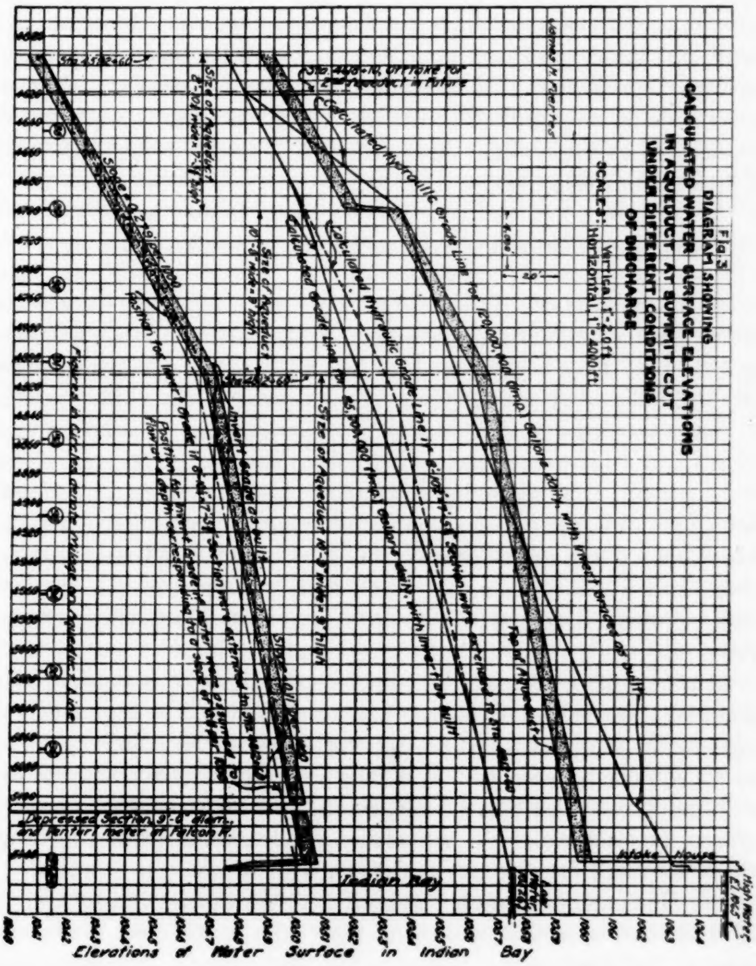
Fig. 3
DIAGRAM SHOWING
CALCULATED WATER SURFACE ELEVATIONS
IN AQUEDUCT AT SUMMIT CUT
UNDER DIFFERENT CONDITIONS
OF DISCHARGE
SCALES: Vertical 1"=2.0 ft.
Horizontal 1"=4000 ft.
Indian Bay
Intake House
Elevations of Water Surface in Indian Bay

smallest is between miles 23½ and 32½, which is on a slope of 1.537 feet per 1000 feet (8.11 feet per mile, the steepest slope on the whole line) and is 6 feet 4¾ inches wide and 5 feet 4¾ inches high. From Mile 17 westward toward Winnipeg the ground falls off in elevation so that to deliver water into the Winnipeg reservoir, the aqueduct must run under pressure, and here, again, enter some interesting hydraulic problems. All the aqueduct, above described, that is from Mile 17 eastward for 80 miles, has a discharging capacity of 85,000,000 gallons daily, a quantity far in excess of the needs of Winnipeg for many years.

HYDRAULIC GRADIENTS AT WINNIPEG END OF AQUEDUCT

General conditions. The plans as worked out for this western end are based on construction details such that quantities up to 28,500,000 gallons daily may be delivered by gravity through the system into the McPhillips Street reservoir in Winnipeg at elevation 769.5, which is high water level in that reservoir. That is, from the western end of the gravity flow aqueduct at Mile 17 to Deacon, at Mile 13, there is a depressed section, having a diameter of 8 feet, followed by 49,900 feet of 5 foot 6 inch lock-joint pipe, reaching to Red River in Winnipeg; then a 5 foot diameter cast-iron-lined inverted siphon under the river, and a 4-foot lock-joint reinforced concrete pipe 11,400 feet long through the streets of Winnipeg to the McPhillips Street reservoir.

As a part of the system, but not for immediate construction, it is planned to provide at Deacon, where the 8-foot and 5½-foot concrete pipe sections join, a reservoir holding 250,000,000 gallons to serve as an equalizing basin and as a safeguard against temporary interruption of flow from Shoal Lake, should this be necessary to permit repairs or cleaning of the aqueduct. The natural ground level at the site chosen for this large reservoir is at an average elevation of about 774.5 and the water level will vary in service from 791.0 to 797; in other words, it will stand from 16.5 to 22.5 feet above the level of the surrounding prairie. The higher level represents the condition when about 20,000,000 gallons daily is going by gravity to the McPhillips Street reservoir, and the lower when about 115,000,000 gallons daily is leaving the reservoir and is distributed to the various communities forming the water district, 35,000,000 going to the bank of Red River by gravity and being boosted thence

through the 48-inch pipe line to the McPhillips Street reservoir, and the balance being distributed to the city and its associated communities from other points.

In order properly to consider the hydraulic problems involved in the west end of the aqueduct, from the Deacon Reservoir to Winnipeg, it will be necessary to say a word about the quantities of water required by the different communities forming the district, the rates at which it must be supplied, and the obligations of the District to its component parts in the matter of purveying the water.

Essential conditions in Greater Winnipeg water district act. 1. The corporation is required to furnish water "in bulk" to the several communities forming the district. The context of the Act discloses that the words "in bulk" mean quantities of water at ground level rather than at city street main pressures. In other words, while the District is not authorized to pump water into the street mains of any of the municipalities, it is commanded to furnish water to any one or all, in such quantities as they may severally or collectively need from time to time, by gravity if possible, and, if not, then by pumping to sufficient head to cause the water to flow from the nearest convenient point on the District's main conduit to the nearest point at the corporate limits of the municipality in question; however, this Act specifically states that the main conduit shall terminate at a point adjacent to the McPhillips Street Reservoir in Winnipeg.

2. Any municipality having water conveyed to it from the District's main conduit through a special connection and branch conduit, shall pay the District annually half the interest, sinking fund, maintenance and operation costs in respect of plant and construction required to convey water from the main conduit of the corporation to the municipality in question.

3. Any municipality having a system of water mains may deliver water through these, in bulk, to another municipality.

4. The corporation is not required to build a main to deliver water to any municipality when another municipality is willing and able to deliver it an adequate supply, but may be required to later build such a main, if the aforesaid arrangement is terminated.

5. The corporation may bear in whole, or in part, the cost of extending the existing mains of any municipality to the boundary of another to avoid the cost of direct mains from its works.

Population growth and water consumption requirements. As a basis for the plan for distributing the water to the different municipalities

therefore, the quantities of water required by each, and the rates at which it may be needed, now or in the future, are of vital importance.

In tables 3 and 4 the populations are based on the actual populations at the time of the formation of the District, extended in proportion to their areas at certain assumed increasing densities of population per acre, based on a study of the rates of increase in the older communities.

TABLE 3

Estimated populations and rates of average daily water consumption in millions of gallons in the communities forming the Greater Winnipeg water district

COMMUNITY	1920		1930		1940		1950	
	Population	Consumption	Population	Consumption	Population	Consumption	Population	Consumption
Winnipeg	218,000	16.13	305,000	25.31	397,000	36.52	490,000	49.00
Fort Gary	8,000	0.59	17,000	1.41	29,000	2.67	42,000	4.20
Assiniboia	13,000	0.96	27,000	2.24	43,000	3.96	61,000	6.10
Kildonan	6,000	0.44	13,000	1.08	22,000	2.02	35,000	3.30
St. Boniface	20,000	1.48	45,000	3.74	85,000	7.82	134,000	13.40
St. Vital	4,000	0.30	8,000	0.66	13,000	1.20	20,000	2.00
Transcona	7,000	0.52	17,000	1.41	30,000	2.76	45,000	4.50

TABLE 4

Estimated population and rates of daily water consumption in millions of gallons in the entire Greater Winnipeg water district

YEAR	POPULATION	AVERAGE DAILY RATE	MAXIMUM DAILY RATE	MAXIMUM HOURLY RATE	EXTREME HOURLY RATE INCLUDING FIRE SERVICE	REASONABLE MAXIMUM HOURLY RATE INCLUDING FIRE SERVICE
1915	207,000	13.65	19.10	31.52	44.12	37.50
1920	276,000	20.49	28.59	45.15	60.15	51.13
1930	432,000	35.85	50.18	75.24	93.24	80.25
1940	619,000	56.95	79.73	113.17	133.57	117.54
1950	825,000	82.50	115.50	159.23	181.43	159.23

The necessity of considering fire drafts in these estimates arises from the fact that, although the city now has a high pressure fire service, in future the fire pumping engines will be supplied with water from the new aqueduct, and the aqueduct system will respond to the demands as occurring, the same as though the water were to be pumped from the McPhillips Street reservoir.

The foregoing estimates were intended for the purpose, only, of forming a basis for the proportioning of the sizes and capacities of the future water supply works the District will have to provide. They may, in individual districts, prove to be more or less inaccurate, but for the purposes intended exactness is not essential, for very considerable variations from the true conditions will but slightly affect the total and will have only a negligible effect on the actual dimensions of the works. They are to be regarded more as aids in fixing upon the ultimate limits then as representing actually the absolute variations to be expected from year to year.

Effect of rate of draft on capacities of pipe line, force mains and pumps. The rate of use of water by a community varies from hour to hour and from day to day; and the actual demands, as may be seen in table 4, may, in the Winnipeg District, amount to two and a half to three times the average daily rate of consumption. To allow for such fluctuations, either the aqueduct must deliver the water as fast as required at any moment, and its actual effective capacity be only from 40 to 33 per cent of its full capacity, or a storage reservoir must be provided near town to supply the water required by sudden heavy drafts.

Deacon reservoir a necessity soon. Where long aqueducts are in service it is more practicable to build large storage reservoirs than to build the aqueduct large enough to accommodate the maximum possible draft for short times, the purposes of these large reservoirs being to allow a full and large supply in response to sudden heavy drafts without necessitating sudden changes in velocities in the pipe line, and to store excess water, not needed during hours of light draft, for use later during heavy drafts.

In an aqueduct conveying water by gravity flow under limited and comparatively low heads, it is not practicable to quickly speed up or check down the velocity of the water in the conduit, as it would be in a closed force main; hence a considerable amount of storage is necessary to permit satisfactory operating conditions.

The difference in rate between average rates of draft and maximum rates is so great, and high rates may prevail so long, that while a day's supply of water in storage is sufficient to balance extremes of draft when the supply can come to the reservoir at a rate equal to the daily maximum rate of draft, it would take several weeks of storage capacity if the water entered the reservoir at the average daily rate of draft to afford the same balancing effect. For this

reason it is quite usual, where the supply can come to the reservoir at the maximum daily rate of draft, to provide at least one day's storage for carrying the peak drafts.

As above stated, the original plans provided for a reservoir of 250,000,000 gallons capacity to be built at Deacon, in line with the above policy. The original plans, however, provided for discharging the water from Deacon to Winnipeg by gravity, up to a limit of 25,000,000 gallons per day, and pumping the water from Deacon to Winnipeg when the consumption shall have exceeded 25,000,000 gallons per day. For this purpose it was provided that the force main from Deacon to Winnipeg should be a 5-foot riveted steel pipe, as far as Red River, then a 5-foot cast-iron-lined tunnel under Red River and then a 4-foot cast-iron pipe from Red River to the McPhillips Street Reservoir. The pumping station at Deacon was intended to pump the water at sufficient pressure to supply the District through the existing street mains in the different municipalities.

Effect of delivery "in bulk" on designs. A reading of the Act, however, discloses that there was no warrant for the delivery of the water, other than "in bulk," and this led to a change in the plans, whereby the water would be delivered in the future, to the different municipalities at ground level, to be repumped by each municipality in accordance with its own needs. This change will limit the variation in rate of delivery from Deacon to the District between the minimum rate and maximum daily rate instead of the minimum rate and maximum hourly rate, which latter would be the case if the pumping were done at Deacon.

The Deacon Reservoir will, therefore, be used for the same purposes as in the original plan, but may be of smaller capacity than originally intended, owing to the fact that part of the original capacity provided, namely, that part required to balance the hourly variations in pump drafts, must be provided nearer the Red River.

For some years there will be no urgent necessity for the large reservoir. Cleaning of the aqueduct will not be needed for several years, and it will be many years hence before the aqueduct between Deacon and Shoal Lake will be called on to deliver water up to its maximum discharging capacity, even at maximum rates of consumption in the District. Nevertheless, with but one aqueduct nearly 100 miles long, it would be prudent to get the reservoir built at an early date.

Taking into account the changes in operating conditions due to the adoption of the plan of delivering the water to the district at ground level instead of pumping it into the mains, the capacity of the Deacon Reservoir can be reduced to 200,000,000 gallons, provisions being made for supplemental storage in the future at a point not far from Red River, sufficient to take care of fluctuations in the hourly rate of pumping, when required, or about 60,000,000 gallons, as will be described later on.

Supply conditions between Deacon and Red River. The plan adopted was to provide at the end of the 8-foot circular aqueduct at Deacon for a connection to the Deacon Reservoir, when built, with a by-pass direct to the 5½-foot concrete pipe from Deacon to Red River, connections being also provided so that water may be later taken into the 5½-foot concrete pipe from either half of the proposed reservoir. All the water will, whether going to the reservoir and thence to the 5½-foot pipe line leading to the Red River or being by-passed directly to the 5½-foot line, pass through a Venturi meter built into the line. At the east bank of the Red River the 5½-foot concrete pipe ends in a surge tank having a 48-inch cast-iron connection to the shaft at the west end of the tunnel under Red River and also a 42-inch cast-iron connection for the suction pipes of booster pumps to be used, later, when necessary, to force the water to McPhillips Street in the quantities required. An over-flow from the surge tank discharging through a 36-inch cast-iron pipe to the river will give relief so that the pressures in the 5½-foot concrete pipe will not exceed certain predetermined amounts when the water may surge therein, due to the starting or stopping of the booster pumps.

There is no valve between the surge tank and the aqueduct, so that by no accident of operation can this connection be closed. The connection between the surge tank and the tunnel under Red River, however, is provided with a valve in the west shaft of the tunnel to enable the surge tank to be cut off; and a valved connection to the tunnel will permit the water taken by the booster pumps from the surge tank to be forced through the tunnel and thence to McPhillips Street, when more water is needed there than will flow there by gravity through the aqueduct from Deacon.

Having thus outlined, in a general way, the conditions to be fulfilled, that is, to deliver to the McPhillips Street reservoir as much water as practicable by gravity and then boost over from Red River surge tank as much as required, bearing in mind the requirements

of each of the municipalities forming the District, the problem became one of fixing upon the proper sizes for the pipe lines and tunnel, the capacities of the booster pumps and the dimensions and details of the surge tank, and to fix upon a plan for the distribution of the water to the different municipalities, in bulk in proportion to their needs, present and future, and to have the works when built such that they could economically be adapted to whatever final plan of future development the District might adopt.

Supply for Winnipeg and its dependents. The city of Winnipeg proper, which, before the introduction of Shoal Lake water supplied Assiniboia, Kildonan and Fort Garry, has at McPhillips Street a 25,000,000-gallon covered reservoir, together with a pumping station used for pumping the water from this reservoir into the city street mains direct. There are no high level distribution reservoirs or standpipes in the system, so that the pumps must respond to all changes of pressure corresponding to a varying rate of draft. This reservoir is large enough to serve until the population shall have grown enough to require an average of about 25,000,000 gallons of water daily, corresponding, for local conditions, to a maximum daily rate of 35,000,000 gallons.

This quantity, therefore (35,000,000 gallons daily), would represent the capacity required in the pipe line from Red River to McPhillips Street to utilize the reservoir to its full practicable capacity. With this condition, the maximum actual rate of pumping of water out of McPhillips Street reservoir might be as great as 60,000,000 gallons daily for an hour or two, the extra quantity of water for these short periods being furnished from the storage in the reservoir. As will be explained later, however, the pipe line from Red River to McPhillips Street reservoir must be able to deliver water at the rate of 50,000,000 gallons per day, as that is the rated capacity chosen for the booster pump.

The McPhillips Street pumping station and reservoir are located in the city of Winnipeg and occupy all the land available for the purpose, so that any increase in reservoir capacity beyond that now available, must, on account of the value of land thereabouts, be had elsewhere. The pumps at McPhillips Street could be increased in number and capacity, and a pumping capacity in excess of 25,000,000 gallons daily could be maintained there by putting the booster pumps at Red River in use to boost over the extra quantity, beyond that which will flow over by gravity, required for peak loads. Careful

studies of this question, however, showed that it would be preferable to definitely limit the capacity of the McPhillips Street station to an average 25,000,000 gallons daily, and establish another pumping station on the east side of Red River for furnishing the extra water in future, through new force mains, when more than 25,000,000 gallons daily should be required in the district now served by the McPhillips Street plant. This plan is simple and has many advantages, among which the greatest is that without sacrificing any of the work to be done under it, at the present time, it could be changed at any time to fit in with any of the other practicable schemes of development. The dimensions of the works, as built, therefore, are determined in accordance with this plan, which involved the least expenditure for the district, both for construction and operation.

Program for development. The practical effect of these assumptions on the design of the works is seen from figure 4, which shows the hydraulic gradient under various conditions of draft.

At the right hand side of the diagram, at station 900, is shown the western end of the last gravity flow aqueduct section, which is 8 feet 9 inches wide and 7 feet $4\frac{3}{4}$ inches high, with its invert at elevation 792.0. The top of the arch would therefore be at elevation 799.40. In order not to have any upward pressure on this arched section the elevation of the hydraulic grade was fixed at 797.5 for the maximum rates of draft at the different critical rates.

A number of studies of different sizes of pipes, with the resulting grades for discharging the water, showed that for the materials to be used a $5\frac{1}{2}$-foot reinforced concrete pipe from Deacon to Red River, and a 48-inch reinforced concrete pipe from Red River to McPhillips Street reservoir, would be most economical for immediate construction and would take care of the consumption until about 1932, at the rates of increase in use of water as given in Tables 3 and 4. This program would require, after that date, the laying of another $5\frac{1}{2}$-foot conduit from Deacon to St. Boniface with provisions at that place for pumping the extra water directly into the street mains of the municipalities both sides of Red River.

At St. Boniface will be required a storage reservoir or pump-well holding about 10,000,000 gallons at the start, to be increased to about 60,000,000 gallons capacity eventually, but by degrees, to equalize the flow through the aqueduct during times of excessive pumping rates; and, for proper service, the smaller capacity of storage, the requisite pumps and the connection from the first $5\frac{1}{2}$-foot

conduit should be ready by about 1926. This part of the ultimate plant, however, and the large reservoir at Deacon, are not yet a necessity and have not been built or provided for, except as to connections with the 5½-foot conduit, which are all in place and ready when needed. At the St. Boniface pumping station the water will be pumped to street main pressure and the cost of the station and the operation be borne by the municipalities benefited, rather than by the District, which has fulfilled its obligation by bringing the water, in bulk, to all its component municipalities.

It will be observed, from figure 4, that the quantity of water to be delivered to the McPhillips Street reservoir is limited to an average of 25,000,000 gallons daily (35,000,000 gallons maximum rate) and that quantities up to 28,500,000 gallons daily can be delivered into that reservoir by gravity. This latter figure would correspond to an average daily consumption of a little over 20,000,000 gallons. The hydraulic grades shown in each case are for the maximum daily rates of consumption.

It will be observed, also, that the head on the 5½ foot pipe may vary from 12 to about 47 feet, the greatest head being for the delivery of about 20,000,000 gallons per day to McPhillips Street by gravity, and the least for the maximum consumption and pumpage at St. Boniface, years hence. After about 1926 the conditions of discharge to McPhillips Street will remain practically the same as to routine operation; that is, an average of 25,000,000 gallons daily will be sent to McPhillips Street, part of it by gravity and the remainder by booster pumping. The lift on the booster pumps in 1926 will be about 40 feet; this will increase gradually to a maximum of 61.5 feet by 1942, when two 5½-foot conduits are in use and the total average consumption in the district about 100,000,000 gallons daily, nearly 50 per cent in excess of the average daily consumption that can be taken care of by the present gravity flow aqueduct between Deacon and Shoal Lake.

Protection of line from strains due to surging. An inspection of figure 4 in connection with the variations in rates of draft between minimum and maximum rates, as given in Table 4, shows that provisions had to be made to protect the portion of the aqueduct between station 900 and Winnipeg against excess pressures due to surging of the water in response to changes in velocities of flow caused by more or less sudden variations in draft, particularly when the booster pumps at Red River would be thrown in or out of ser-

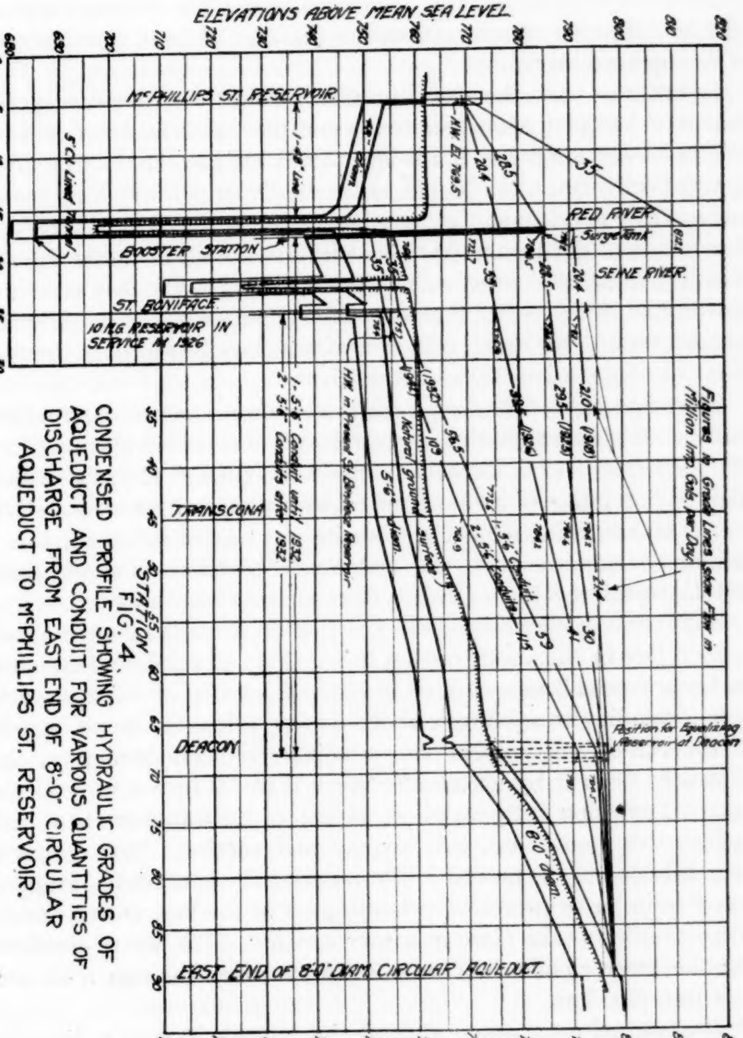

FIG. 4.
CONDENSED PROFILE SHOWING HYDRAULIC GRADES OF AQUEDUCT AND CONDUIT FOR VARIOUS QUANTITIES OF DISCHARGE FROM EAST END OF 8'-0" CIRCULAR AQUEDUCT TO McPHILLIPS ST. RESERVOIR.

vice. These pumps are to be installed with a capacity of 50,000,000 gallons daily, so that pumping can be done at a rather high rate for short times daily rather than be continuous. The storage capacity of McPhillips Street reservoir makes this feasible, and simplifies the pumping machinery.

In order to limit the excess pressures from the above causes, the aqueduct has been connected to a stand pipe, with an overflow with its lip at elevation 785.5, a height somewhat greater than required for the delivery of 28,500,000 gallons daily into McPhillips Street reservoir by gravity, and an overflow has been placed on the gravity flow section at station 900+30 with its lip at elevation 797.60. The last mentioned overflow will protect the gravity flow aqueduct to the east of the 8-foot diameter circular pressure section, and the two overflows, combined, will prevent excessive pressures in the portion between station 900 and Red River.

The surge tank. The surge tank is a reinforced concrete structure, circular in plan, containing a central well, into which the aqueduct discharges, 25 feet in diameter with its top edge at elevation 785.5. This well is inside of and concentric with a second concrete well 32 feet 6 inches inside diameter, with its top carried up to support a reinforced concrete roof, the under side of which is at elevation 294.75, or 9 feet 8 inches higher than the overflow lip.

Both of these circular wells are carried on a circular concrete base some 8 feet in thickness, resting in a grillage of reinforced concrete and heavy steel beams encased in concrete, carried on eight concrete piers extending to solid rock about 34 feet below the inside bottom of the surge tank proper. Surrounding the whole structure, and separated from it by an annular space 2 feet 9 inches wide at the bottom, reducing to 9½ inches at the top of the structure, is a brick facing with stone base, belt courses and cornice. This design is primarily due to the necessity of conserving the heat of the incoming water in order to prevent the freezing up of the first annular space when overflows take place in winter weather. The water overflowing the inner circular well escapes to the river through a 36-inch cast iron pipe line.

The original calculations showed that a tank 20 feet in diameter would be large enough to limit the extreme downward surge to a practicable limit under the assumption of a discharge at the rate of 18,900,000 gallons daily into the tank being suddenly increased to a discharge of 51,000,000 gallons daily by throwing in a booster pump

of that capacity. This proceeding would increase the velocity in the 5½-foot pipe line from 1.47 feet per second, prevailing before the booster was started, to 4 feet per second when full discharge was established. An analysis by a process of arithmetic integration at 20 second intervals of time indicated that the lowest dip of the surge reached an elevation of 751.1, and then started to rise again, at about 6 minutes 20 seconds after starting the pumps. The low point was about 2.3 feet below the final level for a continuous discharge at a velocity of 4 feet per second. The maximum discharging velocity, 4.1 feet per second, was reached in about 8 minutes 20 seconds after starting the pumps. The upward surge from the low point was not followed out in the analysis; neither was the surge from sudden shutting down of the pumps, which would be the maximum to be expected, as these would be entirely checked by the spilling of the water over the overflow lip.

The original plan was for the use of 5-foot steel pipe line from Deacon to Red River and a 4-foot cast-iron pipe from Red River to McPhillips Street Reservoir, and to pump the water from Deacon to Winnipeg at service pressure. As has been pointed out, this plan could not be followed out on account of the conditions in the District's act of incorporation, although, of course, the pipes could be used as recommended. As the new conditions did not require such strong pipes studies were made which showed that reinforced concrete pipes could be substituted for those originally proposed, and a new plan of operation be evolved which would reduce the cost of construction and operation while at the same time providing for the delivery of larger quantities of water to the McPhillips Street Reservoir by gravity. This change was submitted to Dr. Hering and Mr. Stearns, of the original Board of Consulting Engineers, and had their approval.

General description of completed works. Reviewing the subject, it will be seen that the works constructed include the following:

1. Intake at Indian Bay.

2. Venturi meter in the depressed section under Falcon River, about a mile from the intake.

3. Gravity-flow aqueduct on various grades, capacity 85,000,000 miles, with a connection at Station 4618+10 (Mile 87.5) for a second aqueduct in the future.

4. Eight-foot circular reinforced concrete aqueduct, built in trench, from Station 900 to Deacon (Station 678+72.5), a distance of

22,127.5 feet (4.2 miles) running in service under heads of from 10 to 30 feet; capacity 85,000,000 gallons daily.

5. Venturi meter in 8-foot pipe line, then offtake pipe to Deacon Reservoir, then 5-foot shut-off gate in aqueduct, then intake of 5½-foot line from Deacon Reservoir, then Venturi meter on 5½-foot line. These two meters, the two offtakes and the shut-off gate in the 5½-foot conduit are all in a stretch of 294.5 feet of aqueduct, measured along its center line.

6. Then a 5½-foot reinforced concrete pipe from Deacon to the surge tank at Red River; total length 49,900 feet, delivering water to the surge tank at Red River at the maximum practicable rate of 59,000,000 gallons daily from Station 900 to the 36-inch outlet connection for Transcona, 56,500,000 gallons daily to the 36-inch outlet connection for St. Boniface and St. Vital, and 35,000,000 gallons daily from St. Boniface to the surge tank at Red River. A 24-inch outlet, also, is provided at Archibald Street, for the Elmwood District and East Kildonan.

7. The Surge Tank at Red River.

8. Tunnel under Red River, with connections at top of the east shaft for the booster pump which will, later, pump water from the surge tank to the McPhillips Street Reservoir in quantities up to 35,000,000 gallons per day, but at rates up to 50,000,000 gallons per day.

9. A 36-inch connection at top of west shaft of tunnel for a future supply of water to the high-pressure fire pumps in Victoria Park on the bank of Red River, where the west shaft of the tunnel is located, and a 24-inch connection for Fort Garry.

10. A 48-inch reinforced concrete pipe from the Red River tunnel to McPhillips Street Reservoir in Winnipeg, with a 34-inch outlet at King Street for Kildonan and a 24-inch outlet for Assiniboia at Arlington Street. In this pipe line a Venturi meter has been placed just before the pipe reaches McPhillips Street Reservoir.

The capacity of the 48-inch pipe line and 5½-foot line, acting together to deliver water by gravity to the McPhillips Street reservoir, is 28,500,000 gallons per day, at the maximum rate of consumption, which will be reached when the total consumption from this center approximates an average rate of a little over 20,000,000 gallons per day. When that time arrives the booster pumps at Red River will be required and quantities representing average daily rates up to 15,000,000 gallons, above the 20,000,000 gallons, will

have to be pumped to McPhillips Street at rates approximating 50,000,000 gallons per day, to keep the reservoir replenished. As has been stated above, when more than 25,000,000 gallons daily is required in Winnipeg, Kildonan, Assiniboia and Fort Garry, the additional quantity is to be pumped from a proposed new reservoir to be built at St. Boniface.

Stop-plank chambers and overflows, as well as blow-offs, have been put at all the principal river crossings, and at the end of the gravity flow aqueduct at Station 900+30, in addition to the large gates in the intake structure, for the purpose of regulating the rate of flow in the aqueduct, as required, and to prevent upward pressure on the roof or arch of the aqueduct in case of accident of any kind.

DESIGN OF AQUEDUCT SECTIONS

Arch designs. The design of the aqueduct sections was influenced by many considerations, among which the most important were:

1. Solidity of foundation.
2. Economy in the use of materials.
3. Weight and character of material suitable for backfilling over aqueduct.
4. Character and quality of the soil as to chemical characteristics and amount of moisture.
5. Depth of penetration of frost in different soils.
6. Elevation of permanent ground water level relative to invert of aqueduct, as affecting the tendency to float, and to change the form of the section after construction.
7. Practicable construction methods to secure the expedition of the work.
8. Range of temperatures of the water.
9. Character of soil as to relative amounts of clay, silica and organic matter in its composition.

The problems of arch design and invert design were influenced by quite different factors. Any arch design, to be strong enough to resist cracking, must be provided with practically unyielding foundations. In the larger Winnipeg arches a deflection of the invert by so little as $\frac{1}{40}$ inch, would crack it, and then the spreading of the feet of the arch following the further flattening of the invert would crack the arch along the center of the roof. Making the arches twice or perhaps even three times the thickness required for rigid

foundations would not have prevented arch cracks under uneven settlement of the footings of the arch. The principle was therefore adopted to design the arches on the assumption of rigid foundations, and secure these, or as near these as practicable under local conditions.

The form of the arch and the thickness of the arch ring were based, in general, on a backfill load 4 feet deep over the top of the arch weighing 100 pounds per cubic foot, the width of the backfill, on top, being in all cases equal to the width of the aqueduct, inside, with a minimum of 8 feet, and side slopes for embankments of 1 vertical on $1\frac{3}{4}$-horizontal. The concrete was assumed to weigh 140 pounds per cubic foot. The stress diagrams for the arches were determined from the above loads, the earth pressure directions varying gradually from vertical at the top, in the center, to a slope towards the aqueduct of 20 degrees from the vertical, at the bottom of the aqueduct, the concrete weights acting vertically downward, the water pressure within the aqueduct, for various depths of flow, acting radially outward, normal to the aqueduct faces, and ground water pressures, when expected, acting normal to the outside surface and inward toward the aqueduct. The limiting lines for the inside and outside faces for the arches were determined from these stress lines by so adjusting these that for all conditions of loading the resultant lines of force would fall within the middle third of the section. The satisfying of this condition provides that there can be no tension in any portion of the concrete section, and therefore no tendency toward cracking of the arch.

The stresses in the B section of the aqueduct, which was used for a little over $6\frac{1}{4}$ miles, between Mile 17 and Mile 23, 10 miles, from Mile 77 to Mile 87, with the exception of two inverted siphons included in the distance, with a combined length of 410 feet, are shown in figure 5.

Under only two conditions did there appear to be a necessity for reinforcing the arch, and these were under road and railroad crossings, and where the aqueduct crosses the Brokenhead slough, where the backfill material was practically all vegetable matter weighing only about 50 pounds per cubic foot. The internal outward pressure of the water flowing through the aqueduct here would tend to produce deformation of the structure, or lifting of the arch off the invert; and reinforcing steel was necessary to tie the structure together as a unit and counteract the bending tendency in the sides of the arch. The stresses in this section are shown in figure 6.

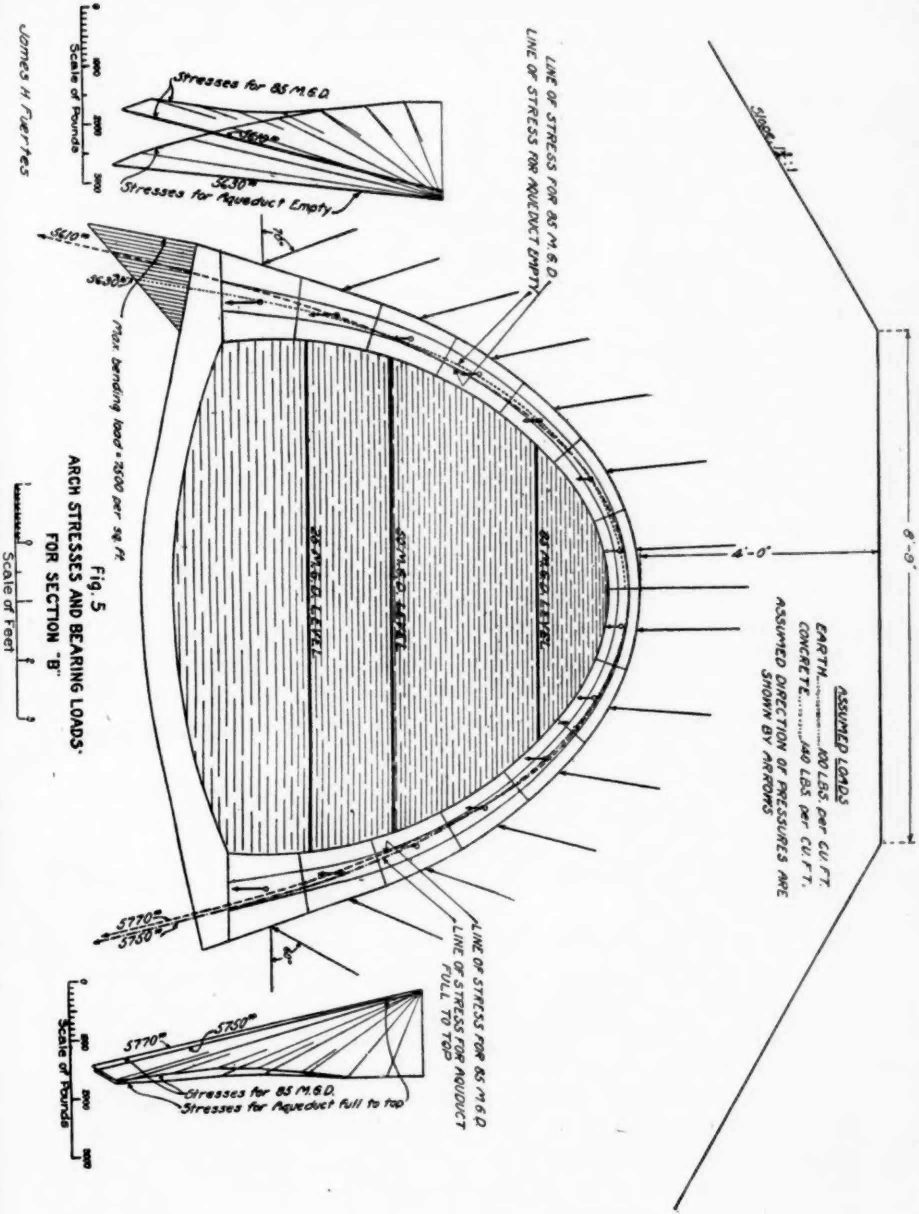

Fig. 5
ARCH STRESSES AND BEARING LOADS
FOR SECTION "B"

As to the possible depth of penetration of the frost, in aqueduct embankments, or its action or effect, some rather surprising facts were noted during the winter of 1915–1916 in investigating this matter. In the prairie section, for instance, about mile 23, it was found, in one place where frost had gone down about 4 feet on the prairie, it was of less depth on the top and side slopes of the embankment over the aqueduct, which, here, was in a rather shallow cut. On digging a trench crosswise of the aqueduct, to examine the condition of the embankment, it was found that the linear expansion, parallel to the surface of the side slopes and tops of the embankment, due to the freezing of the clay soil, had caused the frozen back-fill to rise up where passing over the arch, forming a cavity between the arch and the embankment (supposed to be resting on it). This cavity was about 5 feet wide across the aqueduct, tapering from nothing at the sides to over 6 inches high in the center over the arch.

In another place, in the Brokenhead slough, at Station 2259+33, where the aqueduct is in a 7-foot cut, the frost penetrated the prairie about 4 feet, the bottom of the frozen ground extending horizontally through the backfill and against the arch, where it had frozen fast to the concrete and produced strains, the effects of which were shown in the cracking of the arch on both sides, and its being lifted vertically about ¼ inch; after frost left the ground the arch settled down again in place to its original position. The invert did not crack; the cracks in the side walls were parallel to and 3½ feet above the invert.

These effects were produced, however, before the backfill was entirely completed, and while the aqueduct was empty, during a very cold winter. Practically no expense was necessitated for repairs.

Temperature in aqueduct. Taken as a whole the depth of backfill chosen was sufficient, but not too much. Continuous observations throughout the whole length of backfilled aqueduct during the cold winter referred to showed that the temperature of the air within the aqueduct, except in the immediate neighborhood of open manholes or connections, remained around 35° to 37°F. until the advent of warm weather.

In most places frost did not penetrate to the arch until spring, and then showed in a band of hoar frost of varying widths in different places, sometimes being only a foot or so and sometimes extending down to within a foot of two of the bottom of the arch on each side. The muskeg covering was quite efficient as frost protection,

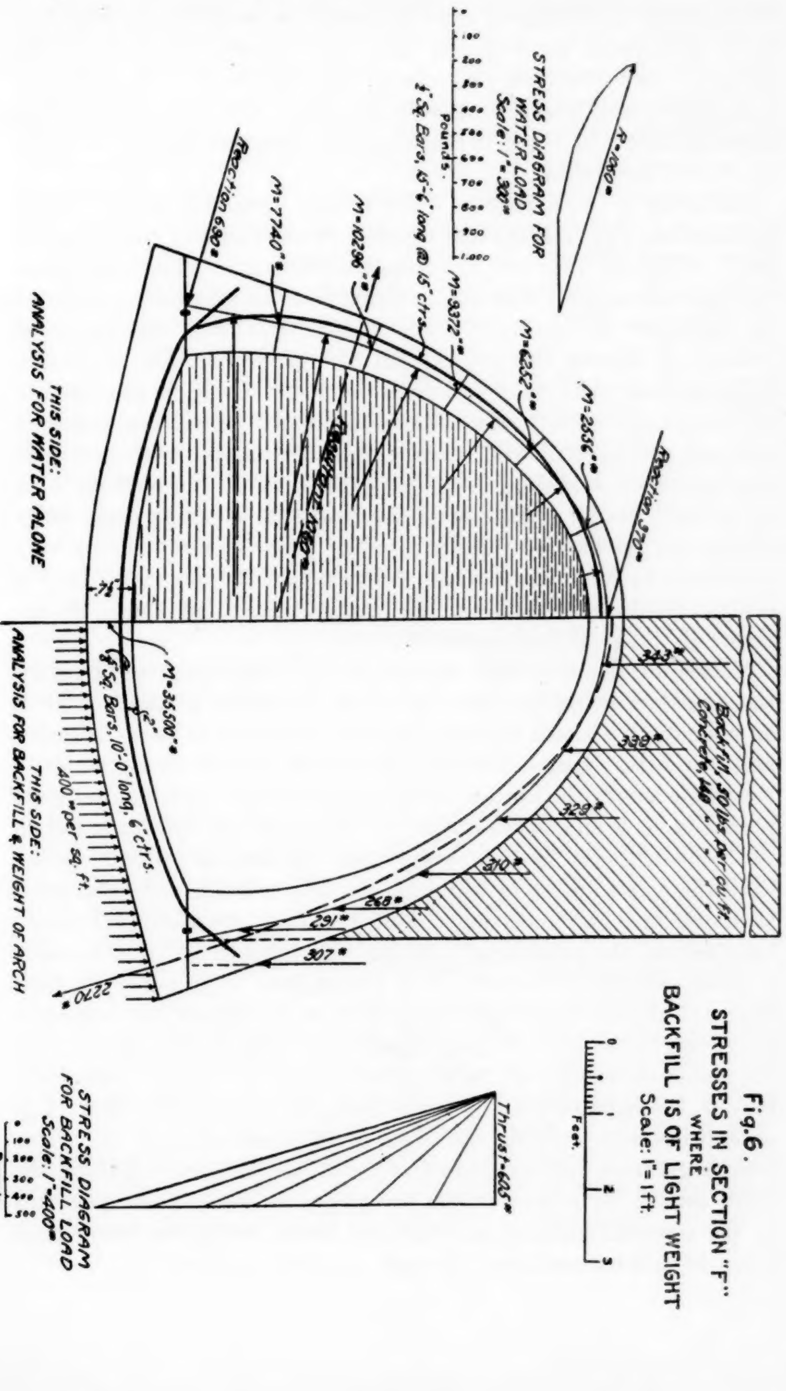

Fig.6.
STRESSES IN SECTION "F"
WHERE
BACKFILL IS OF LIGHT WEIGHT
Scale: 1"=1 ft.

but required a covering of earth or gravel as protection against fire. Frost penetrated most deeply and most quickly through moist, densely consolidated clay, less deeply and more slowly through sand and gravel, and least extensively and most slowly in the muskeg covering after it had been in place for some time and had had a chance to dry out somewhat.

Adequacy of arch designs. The arches designed on the above assumptions have stood without defect or showing any sign of weakness. They are thin and were made so deliberately and after much thought and study. This was recognized in the estimates submitted in the report of Hering, Stearns and Fuertes, and was discussed extensively during the preparation of that report, the conclusion being reached that every justifiable economy should be practised in the building of these works. The difficulties of putting an aqueduct through 100 miles of wild country, largely swampy, with uncertain and unknown conditions as to foundations were fully realized, and, on account of the great cost of the works for a young community grown and growing so rapidly, needing water to maintain its very existence, to have stood upon the principle of "no risk" in the designs would have rendered the project totally impossible, on account of the great cost involved.

Therefore, knowing that settlement might be expected, but that, the aqueduct being one through which the water would flow as in an open ditch without exerting pressure, there would be no difficulty and but little expense involved in making repairs, for practically the whole length the bottom of the cut-and-cover aqueduct has been placed at such an elevation that the surface of the flowing water in the aqueduct may be kept lower than the natural ground surface until the consumption of water by the District shall have increased to or exceed about 50,000,000 gallons per day. An open ditch would have served the purpose of carrying the water from river crossing to river crossing all the way from Indian Bay to Mile 23. In fact, a scheme like this was suggested in 1883 by Dr. Agnew, who proposed conveying the water to Birds Hill and distributing it from there. The cut and cover section of the aqueduct, therefore, is to be considered only in the light of a lining for a ditch, given an arched section in order to permit covering the ditch for the protection of the water from vegetation and dirt, and from cold, and to permit making the ditch smaller by reason of the smooth walls and bottom provided by the concrete surfaces, on which the water would run faster than it would in a natural ditch through the open country.

Depressed sections. Only at the river and valley crossings is this aqueduct in any respect like a pipe line, and at all such places it has been given a circular form and contains enough steel bands to take all the bursting strains due to the water pressure; and all the distortion strains due to the earth backfill, without straining the steel to more than 10,000 pounds per square inch in tension or the concrete to more than 500 pounds per square inch in compression.

These low limits were fixed after much study of the extension of the steel under tension and the amount of extension the concrete, as made, would stand without showing hair-cracks. While it was not a matter of serious consequence whether cracks appear in the cut-and-cover sections of this aqueduct, as these could be easily repaired, it would have been fatal to allow cracking of the concrete containing steel reinforcement, as the steel would then be exposed to the action of the water and would in time be attacked. As rusted steel occupies more space than clean dry steel, the exposure of the steel would result in the scaling of the concrete away from the steel and the ultimate rupture and destruction of that portion of the aqueduct. The stresses in both steel and concrete were limited, therefore, to amounts which experience gained by experimentally bursting reinforced concrete pipe lines by internal pressure showed to be well on the safe side of the danger limits, and, so far as known, to date, these assumptions have, in practice, proved sound.

DESIGNS OF INVERTS

Foundation conditions. In the design of the inverts for the different sections of the aqueduct it was, of course, recognized that the soil to be expected in the trench bottoms would vary from a semi-fluid mud to solid rock. A large percentage of the total was of boulder clay, hard-pan gravel and rock, or of soft soil on top of solid materials, as above described. The first 20 miles, however, was through a prairie country the soil of which was clay deposited from sea water ages ago, and practically devoid of silt or granular materials. This clay, however, is underlaid with a layer of gravel and boulders resting on the limestone rock some 40 feet or more below the surface. This soil is very peculiar and uncertain in its action under loads, and many serious structural accidents have occurred in Winnipeg and vicinity as a result. Its compressibility is variable, depending on its depth below the surface and on its water content.

It appears that this variability is due, in part, to the action of frost when leaving the ground, and to drainage and evaporation of its water content during the dry summer weather. Frost appears to pulverize and loosen the soil for a certain variable depth; drying cracks it open, sometimes to a depth of a few inches, in little blocks, hard in themselves, but resting on a softer base. Further drying, particularly if covered with humus or soil, sometimes opens up long continuous cracks often up to 3 inches in width at the surface, and extending down several feet; when in this condition a rain will fill these cracks, cause the ground to swell, imprison the water in its mass and force it into the lower strata, maintaining these, more or less continuously, in a condition of semi-fluidity. Often there may be a mat, or raft, of dry soil on the surface, many feet in thickness, resting on lower strata in a state of saturation.

Soil tests taken from a dressed surface, which has had a day or two to dry, may sometimes give bearing values high enough to satisfy any reasonable foundation requirement. The same soil, under other conditions as to moisture will flow, under pressure, like putty. But its worst feature is the condition of continuous, progressive settlement under a constant load, and greater relative settlements in small than in greater depths below the natural surface, under the same load.

Compressibility of clay soil at Mile 13. This is shown in tests made at Mile 13 (Deacon) in April, 1916, and abstracted in tables 5 and 6.

From the tables it is seen that doubling the load per square foot at a depth of 4 feet caused $3\frac{1}{4}$ times the settlement; doubling the load at 5 feet caused $2\frac{3}{4}$ times the settlement; and doubling it at 7 feet depth caused a settlement of $1\frac{3}{4}$ times that under the original load; also, that in all the holes and at all loads the settlement at the end of two weeks was about $1\frac{1}{4}$ times the amount at the end of the first day.

Repetitions of tests at other times showed this same general condition but never just the same total amounts. In all the above tests the ground was free from frost at the depths tested, but was moist and in its natural condition for that season of the year. When dry, on the surface, this soil would be as hard as a good road surface, and could be marked with the heel only with difficulty.

Variability in bearing powers. This variability in compressibility introduced conditions making it impossible to build on it any continuous structure that would be free from cracks when finished.

The variable depth of excavation alone would produce this, as, the weight of the structure being the same, the varying compressibility of the soil at the different depths disclosed in building up an aqueduct on a uniform grade would alone cause a wave to travel lengthwise

TABLE 5

Settlement of clay subsoil under a load of 3000 pounds per square foot

DEPTH OR TEST AREA BELOW NATURAL SURFACE	TOTAL SETTLEMENT IN FEET AT END OF DAY NUMBER												
	1	2	3	4	5	6	7	8	9	10	11	12	13
feet	*feet*	*feet*	*feet*	*feet*	*feet*	*feet*	*feet*	*feet*	*feet*	*feet*	*feet*	*feet*	*feet*
4	0.023	0.024	0.025	0.028	0.028	0.028	0.031	0.032	0.032	0.032	0.032	0.032	0.032
5	0.027	0.021	0.029	0.029	0.030	0.030	0.032	0.024	0.035	0.035	0.035	0.035	0.035
7	0.017	0.018	0.018	0.019	0.020	0.020	0.020	0.021	0.021	0.021	0.021	0.021	0.021

TABLE 6

Settlement of clay subsoil under a load of 6000 pounds per square inch

DEPTH OF TEST AREA BELOW NATURAL SURFACE	TOTAL SETTLEMENT IN FEET AT END OF DAY NUMBER												
	1	2	3	4	5	6	7	8	9	10	11	12	13
feet	*feet*	*feet*	*feet*	*feet*	*feet*	*feet*	*feet*	*feet*	*feet*	*feet*	*feet*	*feet*	*feet*
4	0.077	0.090	0.098	0.101	0.103	0.103	0.103	0.103	0.103	0.103	0.103	0.103	0.103
5	0.076	0.079	0.081	0.081	0.082	0.083	0.084	0.086	0.086	0.086	0.086	0.086	0.088
7	0.030	0.031	0.032	0.033	0.034	0.035	0.035	0.035	0.036	0.037	0.037	0.037	0.037

of the aqueduct, as the backfilling progressed, resulting in transverse cracks and being a factor in the production of the longitudinal cracks. No cracks appeared in the aqueduct, either invert or arch, except under the weight of the backfill or the action of frost on unprotected work. Generally none showed with backfills of 2 feet or less.

These conditions were foreseen and fully weighed before proceeding with the design and construction of the inverts. Cracks were expected in this part of the work.

Further east, soft bottom was also known to exist in a number of locations of short lengths. These were, during construction of the work, either entirely dug out and refilled with water-settled sand and gravel, fills, or else rolled embankments; or pile foundations were used. A very few unimportant cracks occurred in the inverts in a few localities so treated.

Special inverts for varying conditions. In some places, particularly where roads and railroads crossed the aqueduct, the section was thickened and reinforced with steel bars to distribute the weight more uniformly over the entire width of the base, to prevent excessive local settlement. Also, where ground water levels were so high as to tend to float the aqueduct, when empty, and where the backfilling materials were very light in weight, the aqueduct bottom was thickened sufficiently to add weight enough to resist flotation of the tube, and the bottom and arch reinforced, as required, against water pressures from without and within, as well as for backfill loads. All these contingencies were foreseen and provided for in the preliminary estimates of cost. No special allowance, however, was made in these estimates for pile foundations, or other means to secure a rigid foundation for the work, through the prairie section above referred to, as it was the judgment of the engineers making the original report that this would not be necessary.

Settlement cracks. As a matter of fact, troubles from settlement occurred only in about 1.6 miles of the 20 miles of contract 30 of this treacherous country; 0.45 mile of the 17¾ miles of contract 31, 0.2 mile of the 18.2 miles of contract 32; about 0.2 mile of the 16.1 miles of contract 33, and about 0.02 mile of the 13 miles of contract 34 by September, 1916, practically all the cracks having been in the 1915 works. This date is given because subsequently a much more expensive form of construction was adopted to prevent such cracks, the changes consisting of using 8-inch extensions or footings on the edges of the invert, to reduce the maximum pressures to lower amounts on poor foundations, and using a heavy, thicker invert, reinforced with steel, where the ground was less hard than the best.

Repairing cracks. None of the work above referred to as cracked was taken out or replaced. Most of it was repaired by cutting out the cracks to a width of about half an inch for a depth of 2 inches

and packing the cracks with hammer-calked neat Portland cement, put in very dry and tamped very solidly, first washing all loose chips, sand and dust out of the prepared cracks with water under pressure. This method of repair made a very strong tight joint; in fact, the edges of the crack were knit together as strongly as though they had never been broken. Several sections of invert, including portions of these joints, have been cut out and tested. The cement filling, even after having been in place only two or three days, adheres so strongly to the concrete that, when broken, the break was generally in the old concrete, sometimes in the filling, but never at the junction of the two.

Water tightness of repaired work. Also, to test the tightness of the aqueduct after repair, a number of sections were selected and tested. One of these, 270 feet long, was purposely picked out to include the most seriously damaged portion. In this section the invert had several longitudinal cracks, some open $\frac{1}{2}$ inch at the surface, others of smaller widths, and the arch was cracked along the haunch on one side as well as along the top. Bulkheads were built at each end of the section, which was then filled with water to the level corresponding to a rate of discharge of 85,000,000 gallons per day. Daily records were kept of the drop in water-level for some months and the rate of leakage found to be less than 5,000 gallons per mile of aqueduct per day, which for this size of aqueduct, equivalent to a circle 8.1 feet diameter, would hardly be called excessive for a cast iron pipe line with calked joints; the average of the other tests gave a leakage of about one-third that amount.

Cost of repairs. The actual cost of the repairs of the 5300 feet repaired, including labor, materials and incidentals, was less than half a dollar per foot of aqueduct repaired; in fact, about $2,500 in all.

With the greater experience gained both by the engineers and inspectors and by the contractors' men, in securing more thoroughly compacted foundations, particularly along the sides of the inverts, practically all invert cracks could have been eliminated in future work without the use of heavy reinforced inverts.

Types of inverts used. The standard invert used throughout during the 1915 construction program, and until April, 1916, was a concrete slab as wide as the extreme spread of the feet of the arch, from outside to outside, with a curved upper surface, struck to a radius of 14 feet for the aqueduct 9 feet high, the largest size used, and 8 feet

$1\frac{1}{4}$ inches for the aqueduct 5 feet $4\frac{3}{4}$ inches high, the smallest size used. The slab was 6 inches thick in the center and for a certain distance each side, then built on a uniform flat slope to the full width required, and was laid in 15-foot lengths, with crimped copper expansion joints between each slab, in advance of and to form a footing for the arches. During the first year's work it was next to impossible to get the contractors' men to consolidate the foundation properly along the edges of the trench, particularly where the soil was of clay, the temptation being so strong to trim the surface to the exact grade required with sharp shovels, which cut it like cheese and left a fine surface for the concrete. This operation, however, left the top surface uncompacted and capable of considerable compression under moderate loads, and a large percentage of the fine cracks in the invert were due to this fact. Finally it was required that all compressible clay sub-grades, particularly in shallow cuts and where the trench was moist or wet, should be covered with a thin layer of gravel, to be heavily hand-rammed for a width of about 3 or 4 feet along the outside edges. When it is realized that a deflection of only about one-thirtieth of an inch would cause cracks in the inverts, the reason for a hair line crack along the center of the invert when the back fill was put over the arch is easily understood where the sub-grade was simply trimmed with a shovel. The uncompacted soil would at times exhibit that much compression under loads of only 700 pounds per square foot, or about 5 pounds per square inch. Rolling these sub-grades even under planks on a gravel bed was impracticable; it would jelly below the surface and work into waves ahead of the roller; hand compacting by heavy rammers on a thin bed of gravel seemed to be the best means of getting hard enough bottom; and when this was done properly and conscientiously, as it was after the first season's work had shown the contractors and inspectors the results of laxity in this regard, the hair line cracks from this cause were practically eliminated.

The large cracks were due to a different cause, the compressibility of the soil by greater amounts. The only way to avoid these cracks was to reduce the unit pressure under the arch footings by spreading the invert to a greater width, or to thicken and reinforce the bottom of the aqueduct so as to distribute the pressure over the whole bottom width of the invert, as uniformly as practicable.

Testing different types of invert to destruction. In order to have something more than theory as a guide as to the actual distribution

of pressure over the bottom of the aqueduct, to aid in proportioning the inverts for this condition of soil, a series of full-size inverts for the 8-foot 9-inch by 7-foot 4$\frac{5}{8}$-inch aqueduct, made 2 feet in width and laid parallel to each other on the natural soil trimmed to shape, was built at Deacon. There were in all 16 inverts, as follows:

A1, A2 and A3. Standard inverts 6 inches thick in the middle, as used in all aqueducts prior to April, 1916.

B. Standard invert with 3 wood strips $\frac{5}{8}$ by 1$\frac{1}{2}$ by 24 inches, built across the invert in the center and near the ends, where cracking had occurred in the construction work.

C1 and C2. Standard inverts 6 inches thick at center with 12-inch extension on each end, making the out-to-out width 14 feet instead of 12 feet as in the original inverts for this size of aqueduct.

D1 and D2. Standard inverts 6 inches thick reinforced with $\frac{1}{2}$-inch bars 8 inches center to center.

E1 and E2. Standard inverts 6 inches thick reinforced with $\frac{5}{8}$-inch square bars 8 inches center to center.

F1 and F2. Standard inverts 7$\frac{1}{2}$ inches thick reinforced with $\frac{5}{8}$-inch square bars 8 inches center to center.

G1 and G2. Invert of standard width, but 10 inches thick, reinforced with $\frac{5}{8}$-inch square bars 8 inches center to center.

H. Invert of standard width, but 11$\frac{1}{4}$ inches thick, reinforced with $\frac{5}{8}$-inch square bars 4 inches center to center.

K. Standard invert 6 inches thick reinforced with $\frac{5}{8}$-inch square bars 4 inches center to center.

These inverts were built in the regular way and were provided with concrete pedestals about 2 feet high on each side, of the width of the sides of the arches. The loading was done by laying 60-pound steel rails across the top of the two pedestals and observing the settlement of the invert at each side, at the middle and at the quarter points, as the load was increased. The deflections or settlement, as reported, were measured with accuracy below a fine bronze wire stretched tightly across, from posts driven deeply into the ground each side of each test invert.

The load and deflection at each point were recorded as each crack appeared, and from these records a number of important deductions were evident.

Practical deductions from tests. For instance, in inverts C1 and C2 cracks did not appear until the load carried was as great as that required to produce the first cracks in inverts F1 and F2, which were

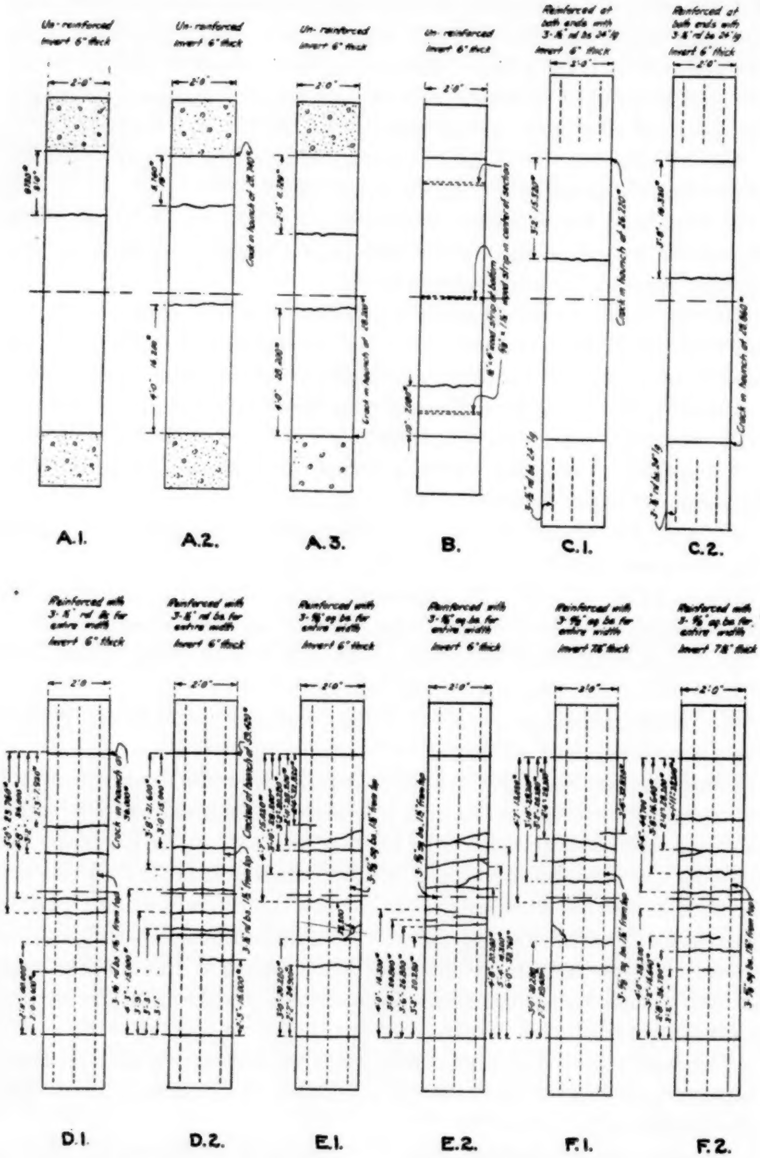

FIG. 7A. POSITION OF CRACKS IN TEST INVERTS AND LOADING CAUSING EACH CRACK

$1\frac{1}{2}$ inches thicker than C1 and C2; and in addition were reinforced with $\frac{5}{8}$-inch square bars 8 inches center to center. In fact, those two thin, unreinforced inverts carried a greater load, without a crack, than any other except those which were heavily reinforced and nearly twice as thick. There was, however, this important difference, that by the time the test load had reached an amount equivalent to the actual load due to the aqueduct and its backfill and water load, a little over 5000 pounds per side per foot, the inverts C1 and C2

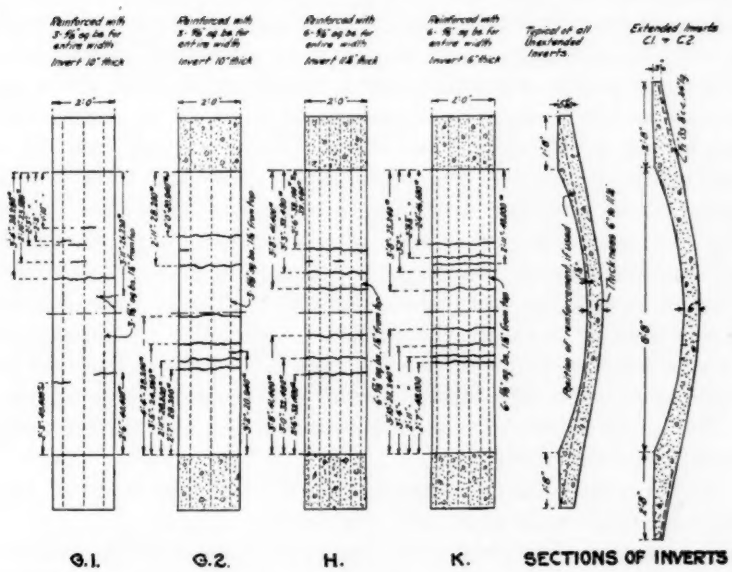

Fig. 7B. Position of Cracks in Test Inverts and Loading Causing Each Crack

had shown but one crack each in the center, which load had to be increased by 50 per cent to cause the second crack to appear at the edge of the invert in each case. Reinforced inverts D1, D2, E1, E2, F1 and F2, showed from 2 to 6 cracks under the same loading as C1 and C2; and even G1, G2 and K showed 2 to 5 cracks across the surface before the load reached the amount normally to be carried by the arch.

In other words, the only reinforced invert that demonstrated its superiority to plain thin inverts C1 and C2 with an extended base was invert H, which was twice as thick ($11\frac{1}{4}$ inches) and also reinforced

with $\frac{5}{8}$-inch square bars 4 inches center to center. Furthermore, the cracks in inverts C1 and C2 could be repaired and the results be made as good as the originals, while the reinforced inverts could not be repaired except at much greater cost, owing to the multiplicity of cracks, and at the risk of further disintegration if acid soil water should penetrate to the steel from below.

The appearance of these 16 inverts, with the location of the cracks and the loads under which they occurred, is shown in figures 7A and 7B.

To prevent further cracks in the invert, a change was made, early in 1916, by adopting for all solid ground, regardless of whether it were compressible or not, a standard invert 6 inches thick at the center with an 8-inch extension on each side beyond the standard invert, and for all questionable foundations, inverts varying from $8\frac{1}{4}$ to $14\frac{1}{2}$ inches in thickness, reinforced with $\frac{5}{8}$-inch bars from 4 to $5\frac{3}{4}$ inches center to center, according to the size of the section. After the tests above referred to were completed and others, also, along different lines, a more conservative program was adopted, as follows:

Types adopted for construction after 1916. Three types of invert were designed, to cover the range of variability in foundations. Type A was for compressible soils; type E for poorer than best but better than worst foundations, and type O for solid foundations.

The invert adopted for solid foundations was the original design used throughout 1916.

For the poorer than best foundations the standard invert of 1915, extended 8 inches on each side, was used.

For the compressible foundations the inverts were the standard inverts of 1915 thickened from 6 inches to from $7\frac{1}{2}$ to $12\frac{1}{4}$ inches, and reinforced with $\frac{5}{8}$-inch bars from $7\frac{1}{2}$ to 9 inches center to center, according to the size of the section. This policy resulted in a considerable saving over the extravagant policy of 1916.

The general designs for these three types, for the B section of the aqueduct (8 feet 9 inches by 7 feet $4\frac{5}{8}$ inches) are shown in figure 8.

While the use of these expensive inverts stopped the occurrence of longitudinal cracks in the inverts, which seemed to be only ones that excited comment and criticism from without, they did not, in the least, stop the formation of transverse cracks in the arches; in fact, these transverse cracks could not be stopped by any means except to put the aqueduct on a pile foundation throughout the country with compressible soil.

The aggregate length of transverse cracks in the aqueduct arches, some caused by temperature, because of delaying the backfilling till too late in the season, but most of them by the plan used by the contractors for backfilling, probably exceeds the lengths of the longitudinal cracks by a very large percentage.

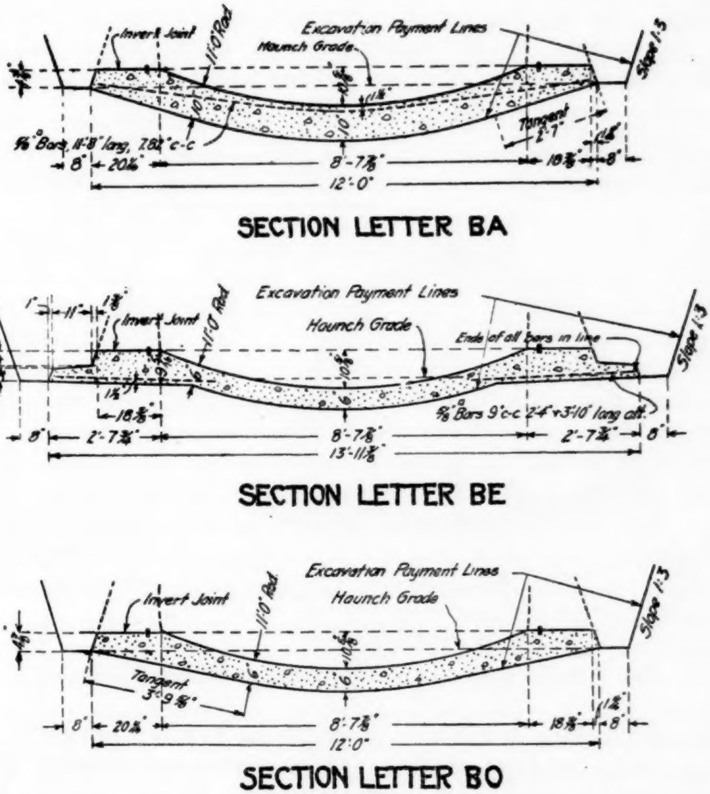

FIG. 8. INVERTS FOR AQUEDUCT 8 FEET 9 INCHES WIDE AND 7 FEET 4⅝ INCHES HIGH

The author attached little importance, so far as the integrity of the work was concerned, to either the longitudinal or transverse cracks, as the cost of preventing these would far exceed the cost of repairs, and the repaired work, being under no strain, would remain in equilibrium in the future.

Precautions to prevent flotation of aqueduct. As to invert designs but one more condition required consideration and that was for use where the available backfill material was of light weight, the aqueduct submerged and the foundation soil porous, or more than a certain depth below grade, varying from 25 inches for the 8-foot 9-inch by 7-foot $4\frac{5}{8}$-inch section to 31 inches for the 10-foot 9-inch by 9-foot section. The problem here was to secure stability and permanence for the aqueduct, full and empty, when the ground water level was above the arch of the aqueduct and the backfill materials so soft,

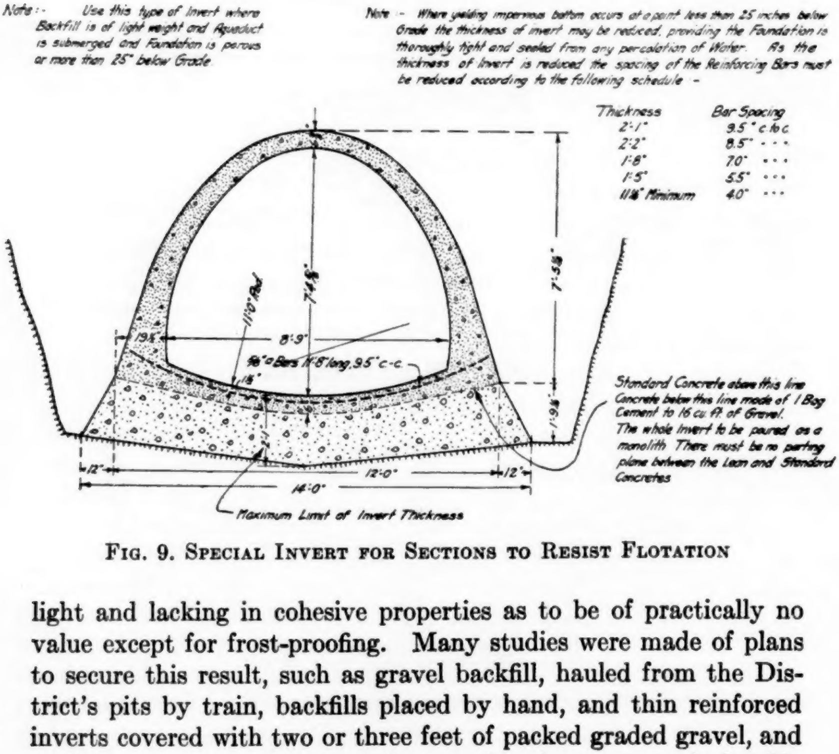

FIG. 9. SPECIAL INVERT FOR SECTIONS TO RESIST FLOTATION

light and lacking in cohesive properties as to be of practically no value except for frost-proofing. Many studies were made of plans to secure this result, such as gravel backfill, hauled from the District's pits by train, backfills placed by hand, and thin reinforced inverts covered with two or three feet of packed graded gravel, and having a concrete paving over the gravel to form a smooth durable invert. The most economical and best plan was the one shown in figure 9.

Acid soil. As has been already mentioned the nature of the processes resulting in the formation of the prairie sections, from Winnipeg eastward to the gravel deposits from the glacial drift about Mile

25, has left the soil impregnated with the salts formed from the evaporation of the sea which once covered this area. The country is so flat that these have never been washed out of the soil, except locally, and there has been, rather, a tendency towards concentration and combination with the soil elements to form compounds which, in the presence of moisture, will combine with the lime in the concrete to form sulphur and chlorine compounds of lime. The sulphur compounds, particularly, are objectionable, because they soften the concrete, if porous at all, and gradually eat into it.

The presence of these salts was found by analyses of many samples in soils at various parts of the line, but not of sufficient concentration to indicate a troublesome action anywhere except certain places west of about mile 30.

Protection of aqueduct. The remedy adopted for this trouble was thorough drainage of the soil to prevent the concentration of the salts locally. The whole aqueduct from mile 17 eastward to Shoal Lake, and the pipe lines from Mile 13 westward to Winnipeg, are protected in this regard. The 4 miles, from mile 13 to mile 17, which is an 8-foot diameter section, with very thick walls, built in place, depressed so low below the prairie that drainage could not be had for the bottom of the trench to any nearby watercourse, and which was the first pressure pipe to be built on the aqueduct, was not underdrained. By an accident of chance the excavation was made when the soil was relatively dry so that the invert could be trimmed exactly to fit the concrete, in the dry, and hence the underdrains leading to sumps that would have been required had the trench been wet, were not put in, and no way exists to drain the soil here without an expensive construction program.

Also, it was assumed that the use of dense hard concrete, carefully placed and with a smooth surface, would resist the disintegrating action of the soil, if the surface waters were properly taken care of, so that there would be no further concentration of the salt locally; and that the actions, if any, would be limited and gradually disappear before damage worthy of attention could be done, by the leaching out of the soil. Along parts of the line, however, in these 4 miles, examinations of the pipe that had been in the ground two years showed, in places, a limited softening of the concrete on the surface, not continuous, as to areas attacked.

REINFORCED CONCRETE PRESSURE PIPES

The premoulded reinforced concrete pipes from Deacon to the reservoir in Winnipeg were built by the Canadian Lock Joint Pipe Company, under general specifications prepared by the District.

The 5½-foot diameter pipe, from Deacon to Red River, 49,900 feet long, was built for and tested in one length to the full head caused by the water in Deacon Reservoir (proposed) with the water shut-off at the Red River, plus a few feet for mild surging. The limiting leakage allowed by the specifications under the above head was at the rate of 115,000 imperial gallons per 24 hours for the 49,900 feet. The actual test showed a safe margin below the requirements.

The 48-inch diameter pipe, from Red River to McPhillips Street Reservoir, 11,400 feet, was built for and tested as a unit, after laying, to a pressure of 40 pounds per square inch. The limiting leakage allowed by the specifications was 9000 gallons per mile per day, and there was, as with the larger pipe, a safe margin below the specified limit.

The pipe was built at Transcona, shipped to the trench and laid by special machinery and methods. The joints were all provided with crimped copper strips, as water cut-offs, every 8 feet for the 5½-foot pipe and every 10 feet for the 48-inch. The methods of manufacture, transporation and laying of this pipe were so well worked out and under control that of the total length of 61,300 feet manufactured (barring the three lengths of 5½-foot-pipe made for experimental purposes by the District, one broken in loading and one purposely broken by pressure in testing), there were only 2 culls on the 5½-foot pipe and 2 on the 48-inch pipe.

DESIGNS FOR SPECIAL STRUCTURES

Overflows and blow-offs. Mention has hereinbefore been made of the use of overflows at all the important river crossings to prevent accidents to the arch section by careless operation of the gates at the intake. In figure 10 are given a plan and section of the overflow at Boggy River, which may be taken as typical.

The main features of these structures are the provision of ample overflow facilities, a blow-out valve, stoplogs to block or regulate the depth of flow in the aqueduct, a boat entrance for taking out boats used for inspection and floating down stream from the next

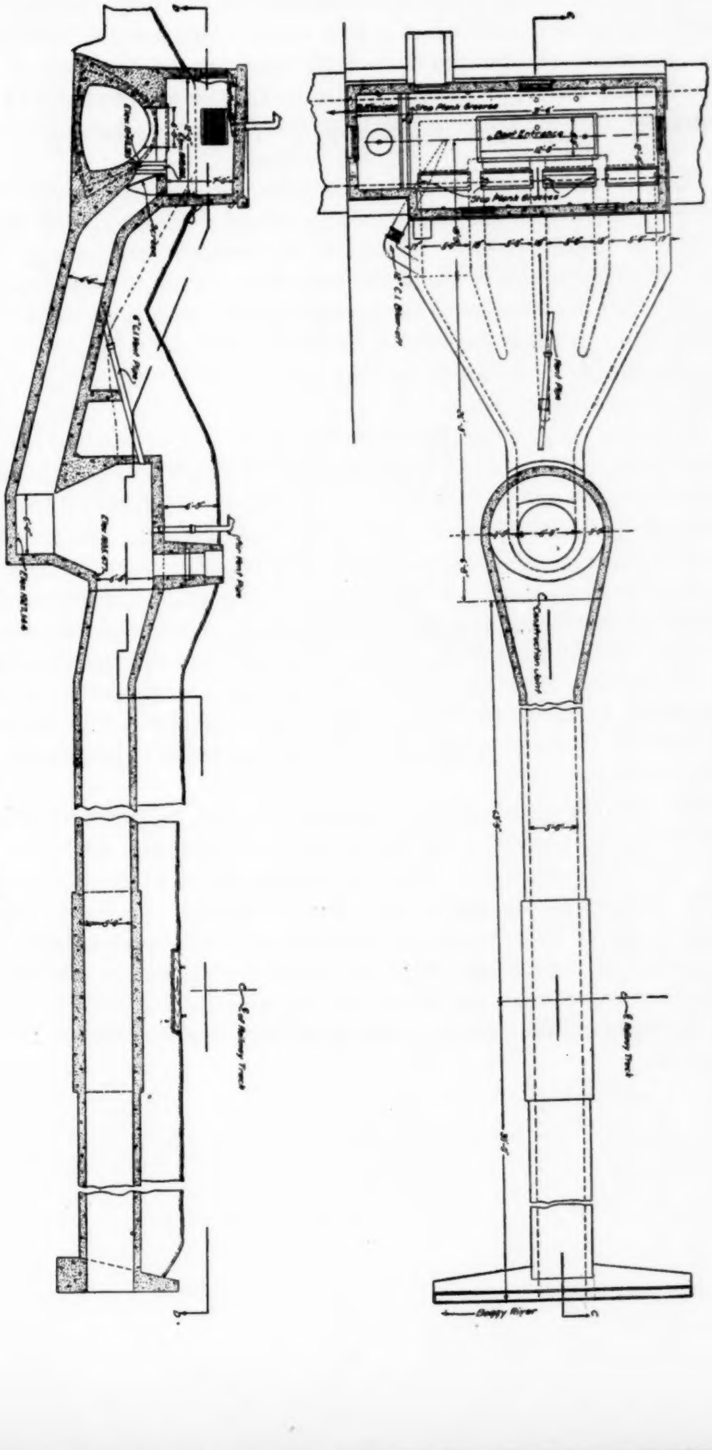

FIG. 10. OVERFLOW AND BLOW-OFF AT BOGGY RIVER

entrance upstream; an overflow channel to the river, arranged in such manner as to exclude the cold outside air from the interior of the aqueduct, and remain open at all times for the discharge of the overflowing water; provisions for balancing the air pressures in the aqueduct, due to changing depths of flow, etc., and a house covering the aqueduct where the overflow is located.

Culverts under the aqueduct. To take care of surface drainage crossing the line of the aqueduct culverts of different sizes were required at various points. Except at the major river crossings, surface drainage is taken under the aqueduct, owing to the elevation of the grade line relative to the ground surface, in an inverted siphon. The details of these structures, shown in general on figures 11 and 12, are similar in all cases, varying only in size or details to adapt them to various locations.

The form adopted allows of cleaning these out, if necessary, from time to time, and provides frost proofing for all parts except the open mouths, which are so designed as to be easily freed from ice forming there in very cold weather The outlet and inlet ends are provided with stop-log grooves to permit blocking out the water for cleaning or removing ice, and have heavy chains hung across the entrance to keep out wandering animals, such as bear, moose, and caribou, as well as cattle in the few localities where there are any running loose in the woods. The top of the conduit, where it passes under the aqueduct, is placed lower than the bottom of the outlet ditch to seal out the cold air which might, if entering, cause trouble with the aqueduct invert.

Miscellaneous small works. Mention, merely, is necessary of the provisions for air valves on the closed pipe lines, and air vents for the valve chambers and other structures where great cold might work damage to the valve, etc., housed therein. In some places, notably on the 5½-foot concrete pipe line, special vents were devised, consisting of a vent pipe with the lower end immersed slightly in a shallow pan of non-volatile oil, an expedient which would permit maintaining equilibrium of pressure without free circulation of the cold air.

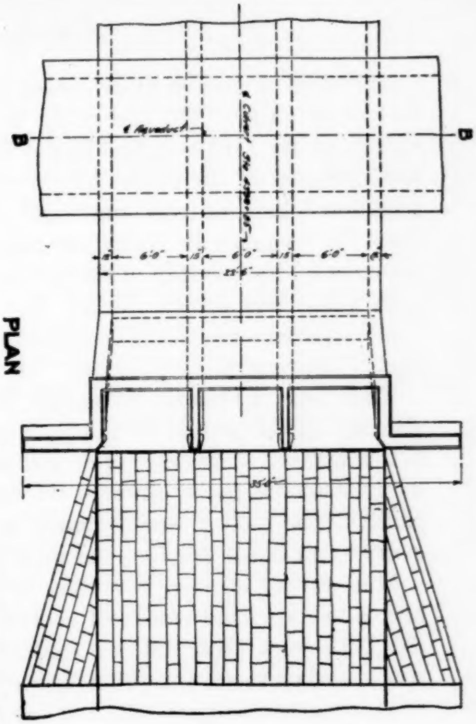

FIG. 11. DETAILS OF CULVERT AT STATION 4399 + 85

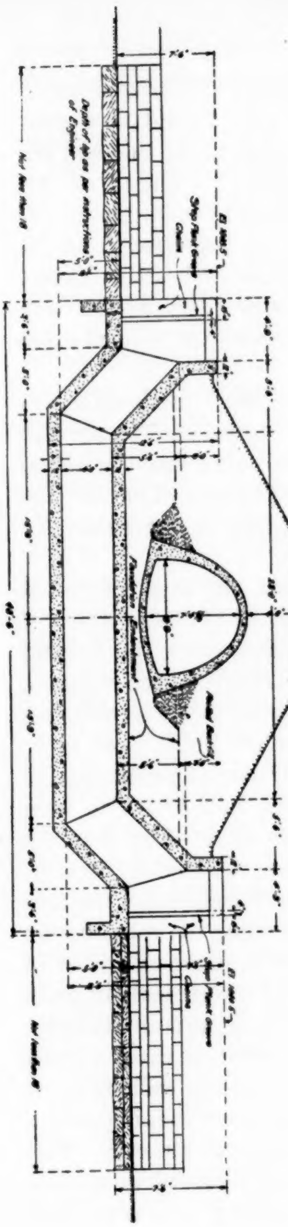

FIG. 12. SECTION THROUGH CULVERT AT STATION 2399 + 85

CONCLUSION

The word of caution incorporated in the report of Hering, Stearns and Fuertes that "The cost of the work will also depend, to a great extent, on the skill exercised in the final location of the line and the establishment of the grades in a proper manner. A margin of half a million dollars, or more, may easily be used up by failure to appreciate the elements in which economies may be practised," can be appreciated by mention of the fact that had the arches and inverts been made but 1 inch thicker than they were there would have been required some 35,000 yards of extra concrete at a total extra cost of approximately $500,000.

There was, therefore, no margin for "slap-dash" methods of designing; all things had to be considered with great care to prevent extravagance. The bases of design had to be sound that the actual works could be pared down to the narrowest margin of excess possible, or the money would vanish in half-million dollar blocks for every extra dollar per foot the work cost.

It is worthy of note, now that the aqueduct has been completed and is in its second season of operation, that all of the portions of the 1915 work that suffered settlement cracks, when the backfill was placed on them, are still in place and in use, having been repaired, as above described; and that the aqueduct, owing to the smoothness of its interior surfaces and the excellence of the finish, both as to lines, grades and uniformity throughout, will, despite predictions to the contrary and despite the cracks, not one half of which were repaired at all, deliver in its present condition more water than the conservative capacity assigned to the scheme in the Hering, Stearns, Fuertes report. Further, the total cost of the construction features of the whole proposition has not exceeded, including the construction railway and all extra work and repairs, the original estimates of Hering, Stearns and Fuertes, the allowances that were made in those estimates to cover extra foundation costs, ditching, swamp drainage, the construction railway and equipment, the probable extra quantities of concrete, excavation, steel for reinforcement, difficulties at stream crossings, and other elements difficult to foresee, having proved sufficient to cover just such contingencies as were encountered. Further, also, these exceptionally fine results were achieved in spite of war conditions, the plans for the aqueduct having been completed early in 1914, and the construction of the railway well advanced

before the war broke out; and the water of Shoal Lake was delivered into the Winnipeg Reservoir in the spring of 1919. But for the fear, locally, of trouble with steam boilers and heating plants as a result of changing suddenly from a very hard to a soft water in the midst of cold weather, the new water could have been made available in the winter of 1918, so far as the aqueduct and pipe lines were concerned.

While the rated capacity of the aqueduct is generally understood to be about 85,000,000 imperial gallons per day, its actual capacity, clean, at the depth of flow giving the greatest discharge, is not much under 100,000,000 gallons per day; even so, however, without the proposed Deacon reservoir in use, the effective capacity of the works, from Shoal Lake to Deacon, will take care, only, of a population using an average of about 60,000,000 gallons per day, and in delivering this quantity there will be risk to the aqueduct unless operated under the care and direction of an experienced man, thoroughly posted, both technically and practically, who knows how to control the flow to prevent surges when the velocity changes from low to high rates, and vice-versa, under the exigencies of service conditions. With the Deacon Reservoir in service the danger to the aqueduct will disappear and its capacity will automatically increase 25 per cent, figured on the average daily demand for water. It is evident, therefore, that as soon as boosting becomes necessary at Red River (which will be done by pumps discharging the water at a rate of 50,000,000 gallons per day), the Deacon Reservoir should be in service; if it is not, the regulation of the flow in the gravity section of the aqueduct, east of Deacon, will present problems of some difficulty, as the frequent slowing down of the flow to an average rate of 1.5 feet and sudden speeding it up again to 4 feet per second, in response to booster demands, will require an expensive organization, or the continuous wasting of a much larger quantity of water at the overflow at the beginning of the 8-foot circular depressed section, at station 900+30, than can be accommodated there without subjecting private property in the vicinity to occasional floodings.

In closing this very superficially descriptive paper dealing with some of the novelties of design in connection with the works of the New Greater Winnipeg Water Supply, the author would be presuming far beyond his province to attempt to apportion the credit for the success achieved. A deep appreciation of his own obligations, however, no less than a sentiment of high personal regard,

pels him to offer here a few words of tribute to all his superiors and associates, and particularly to Thomas R. Deacon, C.E., former mayor of Winnipeg and first chairman of the Administration Board, who was always a most consistent champion of Shoal Lake as a source for Winnipeg's future water supply, and whose keen foresight, inspiring leadership, great energy and tireless exertions turned what seemed a highly desirable possibility into a practical reality; and to the late S. H. Reynolds, C.E., first chairman of the Board of Commissioners, and James H. Ashdown, Esq., Commissioner, whose unselfish devotion to the interests of the undertaking laid the foundations never afterward to be disturbed for the smooth, orderly and properly controlled development of the project.

REPORT OF THE COMMITTEE ON COLD WEATHER TROUBLES

The Committee received 85 replies in response to its questionnaire regarding the cold weather troubles of 1917–1918. The information has been tabulated and is filed with the Secretary. The data thus obtained were both interesting and valuable, but lack the uniformity necessary to make a comprehensive comparison, and the Committee did not feel that the data furnished were such as to warrant the expense of printing.

Extreme low temperatures prevailed throughout the country during the winter of 1917–1918. Unfortunately, the replies to the questionnaire seldom gave temperature where trouble was experienced and did not indicate the close relation between trouble and temperature that one might expect. Trouble was experienced in comparatively high temperatures in some localities and none reported at low temperatures in others, without any explanation of this unusual condition. In few cases was there any mention as to the depth of snow. Nearly all reported an increase in consumption during the cold period.

The data in regard to depth of covering were interesting but not conclusive except in emphasizing the need of giving special study to local conditions. Pipe having a covering of 54 inches of coarse gravel froze while pipe having only 28 inches, laid in wet ground, escaped. The size of the pipe and circulation being the same, it is reasonable to suppose that the nature of the covering material accounted for the difference.

In few cases was there any mention as to the kind of pavement in streets where freezing occurred, and no facts furnished on which to base opinions as to the effects of same.

The question of covering should receive the careful consideration of the local management.

The data in regard to thawing were more complete. Forty-six replies reported in more or less detail the use of electrical current in thawing; 22 reported as to other means, steam, etc. Important details, however, were lacking in many cases. In 53 of the cases

the information was not comparable. Steam was used successfully in a number of cases.

The majority of replies clearly favored the use of electrical thawing and indicated that this method afforded advantages as to both cost and efficiency. The cost data were incomplete, only a few replies giving details as to length or size of pipe, time, etc. The cost for service work ranged from $1.90 to $139 per service; and the time from thirty minutes to two days. In general, it may be said that the expense of thawing a service was under $9.00 and the time required less than one hour. In the majority of the cases reported the current was taken from the local lighting company; fourteen water companies did their own work of thawing; and in five cases the work was done by plumbers. The current used ranged from 5 volts and 150 amperes to 125 volts and 600 amperes. Storage batteries were used to some extent. There is a possibility that the use of electrical current may damage the pipe connections. The Committee recommends that those using this method carefully note the effect with a view of subsequent report to the Association.

Many automobiles were converted into effective thawing outfits, and there are undoubtedly possibilities in this line. The Committee endeavored to obtain from manufacturers specifications for an outfit, low in first cost, light in weight, simple and effective, that could be easily transported by sleigh or auto and operated by gas engine or the auto engine itself. The Committee believes there is a market for such an outfit and hopes that the manufacturers will give the matter earnest consideration. The Committee would recommend a generator operated by a gas or oil engine, mounted on an iron frame, equipped with rheostat, automatic circuit breaker, voltmeter, ammeter, cable, etc., all to be mounted on a trailer or sleigh, and further suggests the use of a current of from 30 to 50 volts and 130 to 400 amperes. There should be a rheostat control to permit increase or decrease in the voltage without interfering with the proper operation of the outfit. By proper manipulation, such an outfit should serve both for service work and reasonably sized pipes and, if properly and carefully handled, would be safe and effective. It was suggested that where high voltage current was available, an alternating current transformer be held in reserve so that if more convenient, current could be taken from the power company. The manipulation in this case should be entirely under the control of the power company. It must be borne in mind that

in the use of any electrical current for thawing there is definite risk and that care and common sense must be exercised.

The greatest cold weather trouble probably comes from frozen services, at least the greatest "kick" comes from this source. The service pipes are seldom laid as deep as the mains and more often with much less covering, exactly the reverse of what it should be. Not infrequently the tap is made near the top of the main and the gooseneck is perhaps 4 to 8 inches above the main. Services should have more covering than the mains and should be carefully inspected, especially if laid by plumbers. A main or service that has once given trouble should be lowered to prevent a repetition of it.

A frozen service is more or less inconvenient but a hydrant out of commission creates a serious risk. Hydrants should receive special attention and every hydrant on the system should be inspected late in the fall. The drip should be open, if the hydrants are set so they can drain. If set in water, the drip should be plugged and the hydrants pumped out. Those on dead ends should be packed. Where it is necessary to take these precautions, hydrants should be inspected frequently. There should be a rigid rule to prevent unskilled or curious people opening hydrants during cold weather "to see if water will run," a not infrequent cause of trouble.

Frozen hydrants were reported successfully cleared by steam, hot brine, calcium carbide, alcohol or by building a fire about them. No information was given as to the effect of salt or carbide on the valves. Alcohol was reported as giving satisfactory results but it is rather expensive and since the Volstead Amendment might be considered unsafe, not, however, as unsafe as the practice of building fires. Ordinary hot brine will do good work. If the trouble is in the branch, salt and carbide are said to be effective.

To the question, "If chemicals were used, give commercial names, etc." the questionnaire brought no response but on the contrary several inquiries as to what the Committee had in mind. It had hopes that some genius might have discovered an inexpensive mixture that would prevent freezing and not injure the valves and working parts of the hydrant.

The operating man must study the conditions in his locality and solve them as a local problem and not be governed too much by what may be good practice in other localities. This refers not especially to cold weather troubles, (which after all are of comparatively small moment) but to the operation of the entire plant. The actual

number of services, hydrants and mains frozen is a very small percentage of the whole, and indicates that water works in general are as efficiently managed as other businesses. The Committee has data furnished by the replies as to the number of frozen meters, services and hydrants but doubts if the average is indicated; and any conclusions therefrom might be misleading.

The Committee feels that it has, so far as was within its power, accomplished the work for which it was appointed, and respectfully asks to be discharged.

Respectfully submitted,

CHARLES R. BETTES,
Chairman.

REPORT OF THE COMMITTEE ON OFFICIAL STANDARDS OF WATER ANALYSIS[1]

Your Committee on Official Standards of Water Analysis has endeavored to go over the available material on the subject of water supply standards, with particular attention to experience with standards as proposed in America, and has also endeavored to obtain information relative to the opinion of representative workers on the general applicability of standards, especially that standard known as the United States Treasury Department standard for water supplied by common carriers to passengers in interstate traffic.[2]

The difficulties in the way of a fully successful general standard, based upon the results of analyses alone, seem to your committee to be insurmountable, at least in the light of our present knowledge. It is easier to conceive of a standard for the operation of a water plant, or for a series of similar sources in a very narrowly restricted area, but even then, if the analytical standard is made rigid enough to be significant, provision must be made to apply it with discretion.

The chief value of fixed standards lies in the simplification of the administrative control of water supplies, in the preparation of guarantees of filter performance and in the facilitating of the attempt to explain technical details to a non-technical body or to the public. A high standard of relative purity may be required in an arbitrary manner, and in the effort to meet the standard plant operators may bring about a general improvement in water supply conditions, as has resulted from the establishment of the United States Treasury Department standard with its governmental prestige. Any such standard must be used with discretion and with good judgment. In the hands of those who wish to use the standard in a rule-of-thumb manner without taking into consideration all known factors, a fixed standard may become a dangerous weapon,

[1] Submitted to the Executive Committee, June 24, 1920, and ordered printed in the JOURNAL. Discussion is desired and should be sent to the Editor.

[2] Bacteriological Standard for Drinking Water (Approved October 21, 1914) Public Health Reports, vol. 29, no. 45, 2959 (Nov. 6, 1914).

able to destroy the good name of a satisfactorily conducted water supply. The United States Treasury Department standard was originally promulgated in 1914[2] as a hard-and-fast, laboratory-controlled standard, but in 1919[3] it was found advisable to modify it to include other details and to allow for a certain degree of elasticity not contemplated in the original work.

Before any attempt can intelligently be made to prepare a standard, it is obvious that all factors capable of influencing the quality of water supplies should be known, and the likelihood of the result being influenced by outside conditions allowed for.

Among the many factors capable of influencing the analytical work, the manner of collecting the sample, the time in shipment, and the temperatures and light conditions to which the sample is subjected are important. In the analytical work itself must be considered the methods employed, the types of determinations made, those which are significant and those which are irrelevant. The accuracy of the various methods is to be weighed, and the results stated accurately. It is generally appreciated, for example, that the statement, "B. coli absent," is practically valueless, since the matter of the kind of media, the quantity of the water examined, period and temperature of incubation, and the extent of the confirmatory tests are all essential information.

Then, too, it is necessary that a standard procedure be decided upon and generally followed for each method selected, and that the meaning of the values obtained in these determinations to be relied upon, be much more accurately known. This having been accomplished, the question of the relative weights to be applied to the indications of the different results must be settled upon.

As has been shown, the state[4] and the city or plant[5] laboratories are by no means in agreement as to the exact methods to be employed for the individual determinations. And this is in spite of the fact that the Standard Methods of Water Analysis (A. P. H. A.) have been recognized for years.

[3] Amendment No. 8 to Interstate Quarantine Regulations of January 15, 1916, amending Section 13 of above regulations. Amendment dated July 14, 1919.

[4] Morse and Wolman, Journal of the American Water Works Association, vol. 5, no. 3, 198 (September, 1918).

[5] Hinman, Journal of the American Water Works Association, vol. 5, no. 2, 133 (June, 1918).

The present Standard Methods of Water Analysis are prepared by committees of the American Public Health Association with the assistance of committees of the Society of American Bacteriologists, the American Chemical Society, and the referees of the Association of Official Agricultural Chemists. The work was begun about 1895, earlier work having been done by the American Association for the Advancement of Science.

While committees of the American Water Works Association do not coöperate in the preparation of the report, the committees chosen by the other bodies have included members of this organization. Of those who have worked on the preparation of the various editions of the Standard Methods, A. P. H. A., fourteen are members of this Association at the present time (list of September, 1919). These are Edward Bartow, J. W. Ellms, George W. Fuller, Allen Hazen, D. D. Jackson, George A. Johnson, E. O. Jordan, H. E. Jordan, W. P. Mason, W. F. Montfort, Earle B. Phelps, R. S. Weston, George C. Whipple, C.-E. A. Winslow. These men are probably among those most likely to be selected by this organization for similar work, although it is true that they were chosen and their work accepted by another body and one which is not exclusively concerned with the problem of water supply. It must be remembered, however, that that organization is concerned with sanitation and the protection of health, rather than with the economics of furnishing water.

The work of these committees has been good, and has received general recognition. The chemical methods have been, perhaps, more satisfactory than the bacteriological methods, due, of course, to the fact that bacteriology is a very young science compared to chemistry. The chief objections to the latter methods as recommended have been the somewhat rapid change in procedure, the apparent influence of the personal preferences of the committeemen, and the adoption of a few procedures out of accord with the current water works practice. At the present time, while the committee on bacteriological methods does not include any members of this society (list of 1919), its work has apparently involved only the addition of a few of the newer bacteriological methods to the fourth edition of Standard Methods of Water Analysis. This edition is fresh from the press. Of these new processes, the ones most likely to influence the current practice are the recommended change to the phenol red method of titration of media, and the adoption of 0.5 per cent sugar broths instead of the older 1 per cent sugar.

Your committee believes that the objectionable features outlined above are necessarily unavoidable in collections of methods of this nature and sees no valid reason why this organization should attempt to formulate any standard methods of water analysis at this time. Your committee recommends that the Association accept the Fourth Edition of the Standard Methods of Water Analysis (A. P. H. A., 1920) as official.

Your committee would recall your attention to the value of frequent sampling of water supplies, and to the grave possibilities of error when too much dependence is placed on the results of a single examination. It is true that water-treatment-plant operators usually appreciate the advantage of repeated examinations, but small plants and those which supply well waters are apt to rely on examinations at too infrequent intervals.

It is evident that the present tendency in the bacteriological examination of water is to test larger and larger quantities for the presence of the B. coli group, but at the same time to narrow down the group of organisms included under that designation, as well as to further subdivide the group into the fecal and non-fecal type organisms.

Coupled with this tendency is the tendency to reduce the expression of the results of tests for the B. coli group to some short numerical form. There are a number of methods of calculating the probable number of these organisms per cubic centimeter or per 100 cc. They do not usually yield comparative results. The schemes of this sort which are probably the best known are the Phelps[6] and the McCrady[7] procedures. That of Phelps having been used in governmental work and being given in the Standard Methods of Water Analysis is probably the method in greatest use in the United States. According to Phelps' own statement, his method is of no value for single samples, but is useful where long series of results of plant operation are to be evaluated. It has been shown[4] that in order to be accurate within 20 per cent several hundred samples must be examined and their results treated by the Phelps method to get the number of B. coli per 100 cc. Moreover the dilutions must be so chosen as to yield a positive and at least one negative on each series of tubes representing a sample of water. Attention is directed to the massing of samples when the Phelps method is used

[6] Phelps, Proceedings American Public Health Association, vol. 33, 9 (1907).
[7] McCrady, Journal of Infectious Diseases, vol. 17, no. 183 (July, 1915).

and to the fact that it is sometimes undesirable to extinguish individual differences.

Your committee would, therefore, recommend that unless at least 100 samples are comprised in the series under consideration, the B. coli index be not reported, and that where an adequate series is considered, the number of samples in the series, together with the dilutions used, be stated. For smaller numbers of samples, some method which does not represent the same degree of fictional accuracy is recommended. A common method of reporting is to express the percentage of positive tubes for each dilution, at the same time giving the number of samples and the number of tubes of each dilution planted. Another common method is the fractional method, in which the denominator of a fraction represents the number of tubes planted and the numerator represents the number of positive tubes. Obviously the fraction for each dilution-size is given, together with the number of samples.

On account of its governmental prestige, the United States Treasury Department standard has perhaps been more generally accepted in the United States than any other standard. It has become the fixed standard of six states and one state has adopted the standard of the United States Department of Agriculture. This standard is based upon that of the Treasury Department, but includes other requirements based on sanitary chemical analyses, "special significance being attributed to the presence of nitrites, free ammonia in excess of 0.05 milligrams per liter and to an undue amount of organic matter." Other state water laboratories use the Treasury Department standard in a less rigid manner, although when doing work on railroad water supplies it is necessary to follow the governmental requirements.

The satisfaction of the plant operators seems to vary directly with the ease with which their particular plants can meet the standard. This is natural. However, the number of plants of towns over 25,000 which claim to meet the standard 100 per cent of the time is rather surprising. It is probable that many smaller plants cannot do so. A number of the better-informed plant operators remain opposed to the arbitrary standards. In some places the objection is to the count requirement, it being claimed that the bacterial count at 37°C. is liable to rapid increase in water of somewhat elevated temperature, and that in this case, as in others, an exact relationship between the sanitary quality of a water and

its bacterial count is apparently non-existent. The more general difficulty is with the requirements regarding the B. coli group organisms. In many cases where river water is chlorinated, types of gas-formers very resistant to chlorine but giving the test for the B. coli group recommended by the Treasury Department, are found.

While the United States Treasury Department standard was claimed at the time of its promulgation to apply only to the water supply of the trains, and any intention to extend the standard to municipal supplies was emphatically disclaimed,[8] the effect of the condemnation of the supply of a community and the posting of notices in the stations declaring the water unsafe has been to create a local pressure which has in many instances forced the improvement of the local supply. This is a matter of great and far-reaching importance in which it is evident the arbitrary standard of the United States Public Health Service has done much good. To what extent the methods of handling the water and of filling tanks on coaches have negatived the precautions and improvements is not a matter within the province of the committee.

It is probable that the Treasury Department standard has at the same time worked some hardship on some large water plants in the United States, which were producing a safe water as judged by the typhoid rates of the communities. The elasticity provided for in the amendment of 1919 should aid in avoiding such difficulties.

In 1914 the committee of the New England Water Works Association on Statistics of Water Purification recommended[9] classifying water supplies according to the extent of the analytical control of operation as follows:

Class 1: Analyses one or more times per day. Engineering data collected and one or more attendants constantly employed.

Class 2: Analyses made regularly once a week or month by trained analyst. Attendant in charge makes simple tests daily.

Class 3: Irregular and infrequent analyses. No daily tests.

It is obvious that in considering the "structural, environmental and operative" conditions, larger factors of safety should be required for plants operating under class 2 or 3.

[8] Jordan, et al, Proceedings Indiana Sanitary and Water Supply Association, vol. 8, 38 (1914).

[9] Report of the Committee on Statistics of Water Purification, Journal of the New England Water Works Association, vol. 28, 220 (1914).

The importance of the sanitary survey in water supply investigations can hardly be overstated. It is so easily possible to conceive of the submission of selected samples, or of samples that do not represent the usual output of a water plant, or even of a well supply, that the value of the collection of data on the supply, an inspection on the ground, and the collection of samples by an observer trained in these matters is easily appreciated. He may locate possibilities of trouble that although long familiar with the plant, the operators have not seen, and he may suggest easy and effective means of avoiding the danger.

In checking over the results of a recent survey of railroad water supplies in Iowa the following figures were obtained as to the agreement between the examination made in accordance with the United States Treasury Department method and the sanitary survey made by an experienced officer of the United States Public Health Service.

Water supplies in Iowa

SUPPLIES SATISFACTORY	SUPPLIES UNSATISFACTORY		
By both survey and examination	By both survey and analysis	By survey, analysis satisfactory	By analysis, survey satisfactory
per cent	*per cent*	*per cent*	*per cent*
67	18	9	6

More significant than this has been the work by the Division of Sanitation, Minnesota State Board of Health[10] since the Minnesota work covers a longer period and includes the results upon a much larger number of supplies. The results given in the following table of unsatisfactory supplies were based on the investigation of 1119 water supplies, of which 730 were found to be unsatisfactory in their existing condition.

Unsatisfactory water supplies in Minnesota, 1912–1918

	UNSATISFACTORY WATER SUPPLIES	SHOWN UNSATISFACTORY BY		
		Field survey and analysis	Field survey	Analytical results
Number	730	354	338	38
Per cent	100	49	46	5

[10] Whittaker, Journal of the American Water Works Association, vol. 7, no. 2, 278 (May, 1920).

Your committee would therefore recommend to the members of this Association that they insist upon a thorough and complete sanitary survey, to be made, by preference, by the person who is to make the analysis or interpret the analytical findings, before any important report is made upon the plant under their control, whether the report is to be made by national, state or municipal authority, or for the information of the management of the plant. It is further recommended that in the matter of the routine or irregular examinations made by an analyst not regularly on duty at the plant, that the analyst be fully informed of any changes or unusual conditions which have occurred in the interval between his visits to the plant.

Your committee, after full consideration of the question, cannot recommend any series of values as standard values for all classes of waters, nor for the waters of one class throughout America.

In making this recommendation it is not unmindful of the important influence of some arbitrary standards, nor of the possibilities of recent work on the operation of filter plants. In the judgment of the Committee such progress as is likely to be made soon in the matter of standards, is likely to come in the control of water plant construction and performance standards.[11, 12]

The committee would therefore submit the following as its definition, not of a "standard" water, but of a satisfactory one:

A water which is reasonably free from noticeable color, odor, taste and turbidity, which is reasonably free from objectionable salts in solution, which is free from injurious effects upon the human body, and which is produced and distributed in such a manner that its quality is practically certain to be maintained continuously in spite of accidents which can be expected in the operation of the plant.

Respectfully submitted,

Jack J. Hinman, Jr.,
H. A. Whittaker,
Abel Wolman,
E. M. Chamot.

[11] Horton, American Journal of Public Health, vol. 7, 380 (1917).

[12] Wolman, American Journal of Public Health, vol. 6, 1153 (1916), Journal of the American Water Works Association, vol. 5, 272 (1918); Journal of the American Water Works Association, vol. 6, 444 (1919).

GROUNDING THE SECONDARIES OF ELECTRIC LIGHTING TRANSFORMERS TO WATER PIPES[1]

REPORT BY L. A. HAZELTINE[2] TO THE ASSOCIATION'S REPRESENTATIVES ON THE AMERICAN ELECTROLYSIS COMMITTEE

I think it would be quite proper for the committee representing the American Water Works Association to recommend a resolution to the Association removing objection to such grounding. Professor Ganz made a study of this question in connection with a report to a municipal water board, from which the following is quoted:

> From the results of my investigation I beg to say that I do not foresee any damage to the piping system nor any danger to persons that would be produced by permitting the . . . . Electric Co. to ground the secondary wiring from transformers to the water piping system in the City of . . . . provided that the ground connection is made either on the outside of buildings or else on the service pipe in the building on the street side of any pipe connections, stop cocks, meters, etc. I am certain that destructive electrolysis of the water piping cannot be produced by such ground connections. I am also certain that this grounding to the water piping does not endanger lives of people, but on the contrary prevents a high and dangerous voltage from being maintained between ground and any part of the secondary circuit wiring, and in this way affords actual protection against danger to life. Grounding transformers' secondaries to water pipes has been practiced in a great many cities for a number of years, and I do not know of a single case where any damage to the piping has resulted from this practice.

In approving the practice of grounding transformer secondaries to water pipes, the Association should require that the rules of the National Electrical Code should be followed. In my opinion the Association should also require that in cases where several buildings are supplied from the same transformer (or group of transformers) but one ground connection to the water pipe should be made, to avoid an interchange of stray direct current between buildings.

[1] Presented at the Montreal Convention by E. E. Minor, for the Association's representatives on the American Electrolysis Committee.

[2] Specialist acting with the Association's representatives on the American Electrolysis Committee; 511 Fifth Avenue, New York.

STATEMENT BY THE UNITED STATES BUREAU OF STANDARDS

The attention of this Bureau has been directed to the following resolution which has been recommended for adoption by the American Water Works Association at its Annual Convention, June 21–25, in Montreal:

WHEREAS, The grounding of the secondaries of lighting transformers to water pipes promotes safeguarding of life and does not constitute a hazard to the piping system, be it

Resolved, That the American Water Works Association approves the practice of the grounding of the secondaries of lighting transformers to the water pipes, providing that the grounding is done in accordance with the rules of the "National Electrical Code" and the "National Safety Code."

We may say without reservation that the adoption of this resolution will constitute a very great and desirable step in the progress that is being made towards the universal application of safety measures with respect to electrical circuits. It is a step, moreover, that may be freely taken without fear of appreciably complicating the relationships already existing between the public utilities concerned, and that will be of vast benefit to the public.

In the past, fears have been expressed in some quarters that the use of water pipes for the purpose of grounding secondary circuits would result in more or less trouble for the pipe-owning companies, chiefly from an increased danger of electrolysis, and also from danger to employees. With regard to the first it has not come within our observation and experience, extending over a period of years, that there is any substantial foundation for it, except where the work is done in direct violation of the rules of the National Safety Code. Where done properly, on the other hand, any current flow in the pipes would originate in the secondary circuits themselves and would not only be relatively small in value, but would also be alternating and hence of negligible effect as far as electrolysis is concerned, even if present in large quantities. This has been demonstrated many times for the commercial frequencies used in the distribution of power.

The only real possibility of increased danger from electrolysis lies in the fact that through inadvertence or otherwise, a secondary circuit may be ground in multiple to two separate pipe systems, or to different parts of the same system, between which an appreciable difference of potential exists due to stray currents from electric

railways. This may result in increased flow of railway current over the pipes, or from one pipe system to another, but is a condition that is easily taken care of by the rules of the National Safety Code which restrict multiple grounds on any secondary circuit, not only to a single pipe system, but also to parts of the same system between which there are no appreciable differences of potential. Where these rules are complied with there is, therefore, no possibility of increased danger from electrolysis.

As to danger to employees, it should be recognized that it may arise, but the chances of it arising under any circumstances are remote, and are especially so where multiple grounds are used. With multiple grounds, that is, where a secondary circuit is connected to the pipes at a number of places, at one or even more of those places work might be done on the pipes in safety with no more regard to the electrical circuit than if it did not exist. It may also be said that it is the common practice to use multiple grounds, only a small percentage of secondary circuits having single ground connections, especially where grounded to water pipes. Where there are single ground connections, however, it is advisable to guard against the remote chance of danger that may exist by requiring the electric company to disconnect ground wires from service pipes when work is to be done on them and reconnect when the work is finished. This is a reasonable and sufficient requirement and has been in force in several places for years with satisfaction to both parties.

In conclusion, this Bureau wishes to emphasize and urge the importance of the adoption of the foregoing resolution by the American Water Works Association, in that it will help to clear up a situation that has existed for many years and retarded in considerable degree the application of safety measures, namely, the reluctance of water and electric companies to establish more points of contact than were absolutely necessary. As far as strictly inter-company interests are concerned, this may be the wisest course, but in the present instance the interest of the public seems to overshadow all others and call for favorable consideration of the resolution, which we are confident will eventually prove to be the best for all of the parties concerned.

DISCUSSIONS

RAINFALL AT MUSCATINE, IOWA[1]

This paper gives in a concise form the results of a rainfall record kept in conjunction with the water works system, and affords an excellent example of valuable records, the keeping of which it seems proper to encourage. It may safely be stated that wherever rainfall and temperature records have been kept by water works companies or water departments, the results have invariably proven useful to a degree fully justifying the trouble and expense of keeping such records.

A standard United States Weather Bureau pattern rain gage can be purchased at the present time for about \$6.50. Maximum and minimum thermometers cost about \$5.00 each. With such simple and inexpensive instruments, any water works superintendent can obtain records which are certain to prove of increasing value the longer they are continued. In connection with maintaining records of rainfall, the writer has one or two suggestions:

The greatest difficulty in the process is involved in obtaining a correct record of snowfall. It has been repeatedly shown that the ordinary rain gage overflow can, when used as a snow gage, gives deficient results in most storms. A better procedure seems to be to expose a flat snow board, consisting of a thin board, covered with a sheet of white cotton flannel, about 12 inches square, on the ground in some location where the snow does not drift. After a snowstorm, the rain gage overflow can is inverted on the snowboard, and a prism of snow cut out and melted or measured in the usual way. The snowboard is then dried, and exposed on the top of the newly fallen snow.

Another matter worthy of consideration in the keeping of such records is the recording and publication of the number of rainfall days in each month. It makes a great difference as regards the amount of water available for supplying reservoirs from a rainfall of say 4 inches in a month whether it falls in four days with an

[1] JOURNAL, 1920, page 127.

average of 1 inch per day, or whether it falls on ten days with an average of 0.4 inch per day. Water works superintendents connected with either gravity or underground supplies, but deriving their water from large rivers, may not fully appreciate the value which meteorological records may have to them. As a matter of fact, water consumption is closely correlated with temperature and rainfall; furthermore, in the case of waters subject to turbidity and pollution, the condition of the water itself is in many respects dependent on weather conditions. Accordingly, it seems appropriate to suggest not only the advisability of maintaining rainfall and temperature records in conjunction with all gravity and underground water supplies, but also of maintaining such records in conjunction with filtration plants.

In this connection, it seems proper to call attention to the recent organization of the American Meteorological Society, for the purpose, among others, of fostering and stimulating the maintaining and utilization of meterological records, especially rainfall records. Water works men interested in this subject can no doubt obtain membership and participate in the benefits of such work, especially in view of the fact that the Society has expressed a desire to be informed as to locations where additional rain gages are needed. Those interested should address the Secretary, Chas. F. Brooks, U. S. Weather Bureau, Washington, D. C.

ROBERT E. HORTON.[2]

WEIGHTS AND CLASSES OF CAST IRON PIPE

The writer is grateful to the Editor for calling attention to a failure to clearly express his thought in the discussion of weights and classes of cast iron pipe.[3] The question of different sizes was not intentionally introduced as the important feature.

A bridge may give way and thereby influence conclusions but it would be unfortunate if engineers ceased their efforts to secure a consistent relation between the several members of a bridge truss or of any structural work. For this purpose factors of safety are often used.

[2] Consulting Hydraulic Engineer, Voorheesville, Albany Co., N. Y.
[3] JOURNAL, May, 1920, page 366.

The American Water Works Association has the credit of determining, fixing and publishing the stresses that should not be exceeded in mains under water pressure. Since with low heads the walls of a pipe may be too thin to admit of transportation and safe handling "Class A" for heads of 100 feet is given a high factor of safety against breakage and this becomes a high factor of safety against bursting pressure. As the specification provides for tensile strength in the metal used for pipes it seems reasonable to use tensile strength factors for the pipe itself.

In these brief notes let only one size, (12 inch), and but one quality of iron, (tensile strength, 18,000 pounds) be considered. For 12-inch inside diameter pipe, "Class A" shows a factor of safety of 37 against bursting pressures. But for the risk in handling the pipes that factor might safely be less than half as great. "Class B" shows a factor of safety of 21 for the specified head of 200 feet. It is fairly evident that if a pipe can be transported and safely laid where the head is to be 100 feet the same pipe can be transported and safely laid where the head is 200 feet. It is also clear that if we could discover an American Water Works Association authority for using 12-inch pipes under a factor of safety of 18 instead of 21, "Class A" pipe could be substituted for "Class B", and "Class B" would go out altogether. That would be a marked advantage.

Skipping over one class for a moment, consider "Class D" for a head of 400 feet. Transportation and laying do not call for special increments in "Class D" pipes. The factor of safety for "Class D" is 13. Now it may be said that if a factor of safety of 13 is suitable for 400 feet head, a factor of safety of 18 is proper for 200 feet head, and "Class B" does go out. There is still another class, "Class C." If "Class A" pipe be substituted for "Class C" pipe the factor of safety with 300 feet head would be 12.46. It might with general consent be said that if a head of 400 feet demanded a factor of safety of 13, a distribution system under 300 feet head would be reasonably secure with a factor of safety of 12.46. Then there would be two classes instead of four classes. "Class A" for pressure up to 300 feet and "Class B," as the present "Class D" would then be called, for service up to heads of 400 feet, and except for the thickened walls to prevent breakage in laying "Class A" the factor of safety would be fairly uniform throughout. Besides this conforms to average experience. No one would hesitate to lay a main under a head of 300 feet when assured that the factor of safety was

twelve, that the metal was of good quality and that the pipe had been thoroughly tested under a pressure of 300 pounds to the square inch.

It is true the figures used above are those of the specification. If correction is to be made for irregular thickness, rust and the like, it is well to note that the correction would not lead to heavier than "Class A" weights for pipes under 100 or 200 feet head, but would need to be applied to pipes of 400 feet head where the safety factor now is least.

The specification of the New England Association, unchanged for nearly twenty years, seems far more creditable and satisfactory. From it such weights as may be desired can be selected and the selection can depend upon the location, character of service required, anticipated future changes and other considerations, all of which should be better understood by those in immediate charge than by pipe founders hundreds of miles away, or by agents who are provided with contradictory figures.

It may be noted that the New England specification invites the placing among the rocks and boulder-clays of that district 12-inch pipes that are lighter by 60 pounds per length than the lightest pipes specified by this Association. And in many cities pipes lighter still for gas mains have been safely laid, perhaps at less depth in many cases but even so in greater danger from traffic.

There would be some advantage in designating weights by localities; "Chicago weights" where filled in streets are full of acids. "Rural weights" where mains may not be disturbed in a century. "Subway weights" where subways are to be built under the mains as soon as they are laid, but even these items could be outlined under the title of General Information.

H. F. Dunham.[4]

EFFECT OF REAGENTS ON FLOCCULATION

Mr. Smith in his paper,[5] "Removal of Clay and Silica from Water," has presented a very interesting phase of the varying effects of reagents on flocculation and the results set forth are in direct accord with the most recent views of colloidal chemists.

[4] Engineer, 32 West 40th Street, New York. This letter was dated March 25; Mr. Dunham's letter on page 773 was dated June 10.

[5] Journal, May, 1920, page 302.

Silicic acid is an emulsoid-type colloid of negative charge and it exerts a pronounced protective effect on a colloidal suspension of the same electric sign, as, for example, clay and silica, making them much more difficult to flocculate. This effect can be counteracted by their preferential adsorption of a strong positive ion, such as calcium, but, as pointed out by Mr. Smith, the element of time is an important factor in bringing this result about.

The preliminary treatment of turbid waters with milk of lime by thorough mixing and sufficient period of contact for maximum adsorption is conducive to best results, where this system of dosing is in use.

Moreover, by reason of this preliminary liming the particles are in best possible condition to be immediately acted upon by the force set in motion through the precipitation of the positive colloid $Al_2\ (OH)_3$ and therefore not only a lesser amount of reagent is required but the rate of flocculation is materially increased.

The importance of this rapidity of initial flocculation is often lost sight of, although it has a decided practical bearing on the size of sedimentation unit required.

A slow rate of flocculation means a relatively long period of detention before attaining the maximum rate of settling whereas with treatment conditions right for an energetic kick off, the maximum rate of settling will soon be developed.

The writer believes that many operators have been slow to realize the practical importance of these various considerations and when we bear in mind that the reagents used in dosing represent dollars unrecoverable their most economical utilization merits detailed study and research. Unfortunately there are no set rules which apply to all cases and to meet the varying requirements in the most efficient manner implies individual analysis and systematic experimentation by engineers qualified in training and experience to solve these problems.

WILLARD A. DEANE.[6]

[6] Research Engineer, The Dorr Company, 101 Park Avenue, New York.

ELECTRIC CAST IRON PIPE AND PREPARED OR FACTORY MADE LEAD JOINTS[7]

The article on electric cast iron pipe and prepared joints by Mr. Carson has been read with much interest. It is not believed any doubt exists among engineers that a better grade of iron is possible of procurement and that a better, more uniform, and consequently a stronger pipe can be made. If the tensile strength of the cast-iron can be made higher, it is obvious a "thinner section" or lighter pipe can be cast to withstand the same pressure and with just as high, if not higher, factor of safety because of its assured uniformity.

It is believed by the writer, after a careful personal inspection of the process, that a better quality of cast-iron can be made in the electric furnace than in the old-type furnace. With the rising price of pipe, now over three times as high as it was in 1914, it is fitting that if possible the weight should be reduced, provided the strength can be retained. Tests made under the supervision of the writer have satisfied him the quality of the iron is better; it is more uniform and close grained in structure, there being, as Mr. Carson states, sufficient graphite to allow the pipe to be easily machined.

As for the prepared or factory-made joints, we believe the process as developed by J. R. McWane is sound both in theory and in practice. Certainly, more uniform joints can be secured, and after each is made, it can be closely examined, which in itself is a wonderful improvement and a great point in its favor. The cost of the work can be reduced by a process of "assembling" at the plant, thus standardizing and cutting down a certain amount of overhead expense, provided, of course, that the charge by the manufacturer for such joint is not made too high. At any rate, the writer is firmly of the opinion that much greater uniformity of joints can be procured. Contractors may prefer for a while to pour their joints by the old-fashioned method because this is apparently something new, and like most contractors, they hate to make the shift. As a matter of fact, from the contractor's point of view, this prepared joint is a great deal easier to handle than the old handmade kind, for reasons which should be perfectly obvious.

WEBSTER L. BENHAM.[8]

[7] JOURNAL, July, 1920, page 477.

[8] Johnson & Benham, Consulting Engineers, New York and Kansas City.

MONTHLY VARIATIONS IN BACTERIA IN EFFLUENTS FROM DIFFERENT STAGES OF PURIFICATION PROCESS AT INDIANAPOLIS

A brief summary of the past five years work on bacterial content of the Indianapolis supply, in the various stages of purification, may be of interest. The three tables summarize the information. Briefly the conclusions are:

Bacterial concentration in streams and partly purified water is inversely proportional to the temperature.

The proportion of the total bacterial flora which is of the Colon group is likewise inversely proportional to the temperature.

TABLE 1

Summary of five years' test for bacteria per cubic centimeter, and B. coli per 100 cc. at Indianapolis

	TEST	JANUARY	FEBRUARY	MARCH	APRIL	MAY	JUNE	JULY	AUGUST	SEPTEMBER	OCTOBER	NOVEMBER	DECEMBER
Raw water	20°	4,632	10,743	7,020	1,555	1,297	1,354	585	406	422	1,272	1,014	2,293
	37°	1,433	2,732	2,293	736	966	1,063	453	356	462	616	625	1,017
	Colon	6,637	5,650	9,076	3,458	3,727	1,447	1,036	870	695	917	5,566	2,601
Settled water	20°	2,546	3,594	1,702	381	292	148	147	209	122	179	508	1,544
	37°	627	428	224	131	193	137	152	360	180	125	244	460
	Colon	2,150	1,071	631	182	282	108	156	69	44	51	962	963
Filtered water	20°	420	357	90	29	30	45	85	38	29	24	78	264
	37°	88	50	49	10	16	41	64	48	53	18	19	41
	Colon	389	103	27	4.6	4.5	5.7	8.0	3.6	4.9	5.0	27.3	104
Sterilized water	20°	47	87	5	4	5	6	7	6	5	7	7	71
	37°	16	15	10	5	5	5	5	6	5	6	6	13
	Colon	2.0	0.74	0.49	0.39	0.56	0.37	0.3	0.47	0.32	0.7	0.49	2.1

While the resistance of organisms of the colon type is less than the total bacterial flora, in a water undergoing purification, the sterilization by means of chlorine products exercises a selective action against organisms of this group. In the five years study 3.1 per cent of all 37° growers in the raw water were B. coli. Settling and filtration reduced this proportion to 1.5 per cent and 1.6 per cent respectively. This indicates a certain selective action against, or speaking correlatively, an inferior resistance of B. coli when compared to the total flora capable of growing at 37°C. Ster-

TABLE 2

Comparison of 5 years' total counts at 20°C. and 37°C. expressed in percentages which 37° count is of the 20° count

	JANUARY	FEBRUARY	MARCH	APRIL	MAY	JUNE	JULY	AUGUST	SEPTEMBER	OCTOBER	NOVEMBER	DECEMBER	AVERAGE
Raw water	31	25	33	47	74	77	78	88	110	48	55	44	
Settled water	25	12	13	35	66	93	103	172	147	70	48	30	
Filtered water	21	14	21	35	53	91	75	126	183	75	24	15	
Sterilized water	34	17	200	125	100	83	71	100	100	85	85	18	

Percentage of 37° organisms which are bacteria of the Colon Group (5 years)

	JANUARY	FEBRUARY	MARCH	APRIL	MAY	JUNE	JULY	AUGUST	SEPTEMBER	OCTOBER	NOVEMBER	DECEMBER	AVERAGE
Raw	4.6	2.1	4.0	4.7	3.9	1.4	2.3	2.4	1.5	1.5	8.9	2.6	3.1
Settled	3.4	2.5	2.8	1.4	1.5	0.8	1.0	0.2	0.2	0.4	4.0	2.0	1.5
Filtered	4.4	2.0	1.4	0.4	0.3	0.1	0.1	0.07	0.1	0.3	1.4	2.5	1.6
Sterilized	0.12	0.05	0.05	0.08	0.11	0.07	0.06	0.08	0.06	0.1	0.08	0.17	0.10

Percentage of organisms of the Colon Group which are fecal type (positive reaction to Methyl Red) 3 years

	JANUARY	FEBRUARY	MARCH	APRIL	MAY	JUNE	JULY	AUGUST	SEPTEMBER	OCTOBER	NOVEMBER	DECEMBER	AVERAGE
Raw	55	71	73	74	79	62	61	46	73	71	64	71	66
Settled	63	63	61	66	78	65	48	23	72	66	58	65	61
Filtered	66	80	71	65	70	46	32	23	47	57	58	67	58
Sterilized	70	60	34	31	13	42	25	18	50	57	55	81	56

TABLE 3

Effect of various steps of purification process on bacterial growth evidenced by reduction of 20° growers, 37° growers and colon group

	TEST	JANUARY	FEBRUARY	MARCH	APRIL	MAY	JUNE	JULY	AUGUST	SEPTEMBER	OCTOBER	NOVEMBER	DECEMBER
Reduction by settling and partial coagulation	20°	45	67	76	76	78	89	75	49	71	86	55	33
	37°	66	84	90	82	80	87	70	−1	61	80	61	60
	Colon	68	81	93	95	93	93	85	92	94	94	83	63
Reduction by filtration of settled water	20°	83.5	90	94.7	93.4	90	70	43	82	76	87	85	83
	37°	86	88	92	92	92	70	58	87	71	84	92	91
	Colon	82	91	96	97.5	98.2	94.8	95	95	89	90	97	89
Reduction by sterilization of filtered water	20°	89	75	94.5	86	84	87	91.8	84	83	71	91	73
	37°	82	70	47	50	69	88	92	87.5	91	67	69	68
	Colon	99.5	99.3	98.2	99.2	98.8	99.4	99.6	98.7	99.4	98.6	98.2	98.1
Reduction by entire process	20°	99	99.2	99.93	99.75	99.62	99.56	98.8	98.5	98.8	99.45	99.4	96.9
	37°	98.9	99.5	99.5	99.3	99.5	99.5	98.9	98.3	98.9	99.0	99.0	98.9
	Colon	99.97	99.99	99.99	99.99	99.98	99.98	99.97	99.95	99.96	99.93	99.99	99.92

ilization, however, reduces the percentage of 37° organisms which are B. coli to 0.1 per cent. B. coli unquestionably succumb to chlorination much more completely than the general run of blood-temperature growers.

Of the total number of coli-type organisms present, the methyl red positive, or so called fecal type, survive purification processes, step by step, in increasingly less proportion as the temperature rises.

The B. coli content of the raw water ranges from a minimum of 695 per 100 cc. to 9076 per 100 cc. This minimum figure is higher than the limit set in 1914 by the International Joint Commission. Their paragraph 4 sets a limit of 500 B. coli per 100 cc. for a purifiable raw water. At the time their report was made public, the writer felt that the limit was not set with sufficient information as to actual performances. In his opinion practically no supply in the Central West has a raw water that does not exceed this limit, yet the data of this plant show over a five year average, a range of B coli in finished product from 0.3 to 2.1 B. coli per 100 cc. The later years summarized reduce the maximum figure to less than 2.1.

There is unquestionable and decided need of work in a number of more active laboratories before too broad conclusions are made to certain ideas. This has a particular bearing upon the subject of standard methods of water analyses. It is about time for the American Water Works Association to assert its rights in the matter of methods of examining public water supply.

Harry E. Jordan.[9]

[9] Superintendent of Filtration, Indianapolis Water Company.

IMPORTING LABOR

While the writer appreciates the present demand for labor and the conditions so well expressed in Mr. Ericson's letter,[11] something within rebels a bit at the thought of ever inviting to this country any classes of people without they entertain a strong desire and a plausible plan to remain here and become Americans. The writer has the same feeling in regard to individuals, excepting those who might come in a professional capacity or as visitors. It may be the dwarfed conservatism of age that affects him but it is not easily shaken off.

A hundred years ago this country was in great need of labor. Few, comparatively, thought any harm could follow the importation of animals from Africa. We do not apply the same term now and we do not feel very well satisfied with the commercial venture of that period. The slaves found friends here. That was the first round in the ladder that led up to equal political rights. But during the same interval there was awakened in the cultivated masters a dominant, autocratic spirit that has been referred to in halls of Congress and in colleges also as "a disgrace." That is a mild term. But that spirit as one may now see it illustrated in both the South and North, will become more aggressive and reprehensible with every addition to the numbers of those who serve in a lowly or "degraded" capacity.

"Menial and rough work our white labor will not longer do" cannot be so very different from the work of the white people who carried our country with reasonable success through two or three hundred years of its history in which occurred wars and recovery from wars. The writer will grant this: many white men do not work now in the spirit of the pioneers. The sturdy laborer (white) observed from an office window, takes twelve steps for each half shovel of earth moved one cast from between the rails on Broadway. His eye is on the foreman. His spirit all wrong, made so in good part by the Unions.

If workers would think of what they could make for others or themselves as well as of what they can get, the outlook would be more hopeful.

How refreshing it is in these days to come upon an incident like this as described by John Macaulay:

[11] JOURNAL, July, 1920, page 611.

In 1777 at the age of 23 William Murdoch applied to Boulton & Watt for work. Boulton had nothing for him and was turning him away when he noticed a hat of peculiar pattern in Murdoch's hand. "What is your hat made of?" "Timber, Sir." "What, wood?" "Aye, man." "How was it made?" asked Boulton in astonishment. "I turned it masel in a lathey of ma ain makin." Murdoch was given a place at once and it should be remembered that he made the first steam propelled vehicle in England.

It is true that incidents similar in spirit occur now, but how rarely in proportion to the number of workers. Would the plan of importing Chinese labor tend to insure greater interest in men and work rather than in the results of labor?

When the war ended so unexpectedly, there was apparently every sound reason for the wealthy and for workers in this country to rejoice and in a thankful spirit to work on with greater diligence and to practice greater economy. Instead of that there were unheard-of extravagance and strikes without limit, although each sensible individual knew well that every like demonstration increased the difficulties of the people and of the overburdened administration at Washington. Would the importation of Chinese labor tend to arouse a right national spirit in an emergency like the present? Would it increase and strengthen the bonds of sympathy between the wealthy and the workers? Or between the workers themselves? The writer can hardly think so and it would seem wiser to let some projects wait while the good work goes on of devising methods and machines by which more intelligent labor secures more economical results than were ever known before. An acre of corn should be grown or a cubic yard of granite moved with less expense and labor than now, and preferably by Americans. It seems to the writer that Oriental labor would be in competition with the labor of our own people. Would the American Federation of Labor favor the plan? One further objection is the straight lead into politics.

H. F. Dunham.[4]

SECRETARY'S REPORT FOR THE FISCAL YEAR ENDING MARCH 31, 1920

OPERATING EXPENSES

Office expenses	$675.23
Convention expenses	608.08
Printing and distributing JOURNAL	6,681.91
Committee expenses	1,033.01
Election expenses	225.72
Salary of assistant to secretary	1,200.00
Salary of secretary	500.00
Rent of office	540.00
Insurance, fire and liability	72.40
Salary of editor	1,000.00
Contingencies	192.01
Section expenses	399.84
Total operating expenses	$13,128.20

OPERATING INCOME

Initiation fees	$1,270.00	
Annual dues	7,582.50	
Subscriptions to JOURNAL	446.00	
Advertisements in JOURNAL	2,754.65	
Sale of *Proceedings*	39.00	
Exchange on checks	.98	
Interest on investments	555.04	
Sales of JOURNAL	114.30	
Interest on deposits	108.97	
Typhoid Toll, sales of reprints	2.75	
Authors copies, sales of reprints	62.25	
1916 binding cases	.31	
Hydrant and valve specifications	1.25	
Jordan reports	1.50	
Pipe specifications	37.20	
Uniform accounts	.30	12,977.00
Net loss from operation		$151.20

Office rent paid for one-half year only.

BALANCE, APRIL 1, 1920

54	1919 binding cases	$22.49	
93	1917 binding cases	30.22	
123	1918 binding cases	58.13	
135	Office equipment	108.40	
136	Profit and loss account		$14,552.80
141	Permanent investment fund	11,945.00	
	Balance cash on hand	2,388.56	
		$14,552.80	$14,552.80

Expenditures 1919–20 greater than 1918–19		$4,522.33
Receipts 1919–20 greater than 1918–19		2,318.79
Increased expenses less increased receipts		$2,203.54
Profit from operation 1918–19		2,052.34
Net loss from operation 1919–20		$151.20
Received from redemption of bonds		$2,000.00
Invested in bonds for permanent fund		1,000.00
Investment funds applied to operation 1919–20		$1,000.00
Cash on hand April 1, 1920	$2,388.56	
Cash on hand April 1, 1919	1,603.03	
Increase cash on hand April 1, 1920 over 1919		$785.53
Receipts from sale of bonds used in operation		$214.47
Paid for binding cases on hand for sale and sold but not collected for		63.27
Loss from operation fiscal year 1919–1920		$151.20

FINANCIAL STATEMENT

Cash on hand April 1, 1919	$1,603.03
Receipts	
Initiation fees	$1,270.00
Annual dues	7,582.50
Subscriptions to JOURNAL	446.00
Advertisements in JOURNAL	2,754.65
Sale of PROCEEDINGS	39.00
Exchange on checks	.98
Interest on investments	555.04
Sales of JOURNAL	114.30
Interest on deposits	108.97
Typhoid Toll, sales of reprints	2.75

Authors copies, sales of reprints		$62.25
1916 binding cases		.31
Hydrant and valve specifications		1.25
Jordan reports		1.50
Pipe specifications		37.20
Uniform accounts		.30
Redemption American Foreign Security Bonds		2,000.00
		$16,580.03
Disbursements		
Binding cases	$63.27	
Office expenses	675.23	
Convention expenses	608.08	
Printing JOURNAL	6,681.91	
Committee expenses	1,033.01	
Election expenses	225.72	
Salary of assistant to secretary	1,200.00	
Salary of secretary	500.00	
Rent of office	540.00	
Insurance, fire and liability	72.40	
Salary of editor	1,000.00	
Contingent expenses	192.01	
Section expenses	399.84	
Purchase Victory Loan bond	1,000.00	14,191.47
Balance cash on hand April 1, 1920		$2,388.56

MEMBERSHIP STATEMENT

Active:			
Last report		1,024	
Elected during year		216	
Restored, paid back dues		22	1,262
Losses:			
Resigned		26	
Died		10	
Dropped, nonpayment dues		45	
Transferred to Corporate and Association		4	85
			1177
Corporate:			
Last report	91		
Elected during year	18		
Transferred from Active	3	112	
Losses:			
Resigned	1		
Dropped non-payment dues	4	5	107

Associate:			
Last report	121		
Elected during year	18		
Restored, paid back dues	1		
Transferred from Active	1	141	
Losses:			
Resigned	7		
Dropped nonpayment dues	1	8	133
Honorary:			
Last report	5	5	5
Total membership March 31, 1920			1,422

	Active	*Corporate*	*Associate*	*Honorary*	*Total*
1920	1,177	107	133	5	1,422
1919	1,024	91	121	5	1,241
Gain	153	16	12	0	181

Total number of members received during fiscal year 252 with a net gain of 181. Largest previous net gain 131—1908.

ANNUAL REPORT OF THE TREASURER

Permit me to submit my report as Treasurer of The American Water Works Association for the year ending March 31, 1920.

The funds of the Association are on deposit with The Troy Trust Company, Troy, New York as per order of the Finance Committee.

The receipts during the year were as follows:

Balance April 1, 1919	$1,603.03
Received from J. M. Diven, Secretary	12,939.44
American Foreign Securities Company, notes	2,000.00
Interest on deposits	112.02
Interest on investments	555.04
Total	$17,209.53
Disbursements, as per vouchers, cancelled checks and debit slips	$14,817.01
Balance, April 1, 1920	$2,392.52

The receipts include $2,000 for notes of the American Foreign Securities Company, which became due August 1, and were paid.

The disbursements include $1000 used for the purchase of a United States Victory Bond for the Permanent Fund.

Attached you will find certificate of the Troy Trust Company, showing a deposit of $2,868.32 at the close of business March 31, 1920.

From this balance there should be deducted the following for unreturned checks.

Deposit as per certificate			$2,868.32
Unreturned checks			
V974—Ck. 1144	John T. Williams	$90.00	
979—Ck. 1149	F. A. Barbour	77.20	
984—Ck. 1154	Edward S. Cole	150.00	
985—Ck. 1155	Williams & Wilkins Company	3.75	
986—Ck. 1156	Henry Stowell & Son	147.57	
987—Ck. 1157	Addressograph Company	7.28	
			475.80
Balance April 1, 1920			$2,392.52

The receipted vouchers, cancelled checks and debit slips with the book of the Treasurer are submitted for audit.

The Permanent Fund now consists of the following:

	Par value
4—$1.000 Dominion of Canada 15 yrs. 5% Bonds........	$4,000.00
4—$500 U. S. Liberty 1st, 3½% Bonds..................	2,000.00
1—$1,000 U. S. Liberty 2nd, 4¼% Bond..................	1,000.00
2—$1,000 U. S. Liberty 3rd, 4¼% Bonds.................	2,000.00
2—$1,000 U. S. Liberty 3rd, 4¼% Bonds.................	2,000.00
2—$1,000 U. S. Liberty 4th, 4¼% Bonds.................	2,000.00
1—$1,000 U. S. Victory 4¾% Bond.......................	1,000,00
Par value of permanent fund.........................	$12,000.00

The Treasurer receives no salary and is under $10,000.00 bond as per the order of the Finance Committee.

Respectfully submitted,

James M. Caird,

Treasurer.

Troy, N. Y., 1920.

This is to certify that we have examined securities submitted by the Treasurer and find the same as stated above.

William P. Mason,

Jas. H. Caldwell,

Committee.

Troy, N. Y., April 10, 1920.

This is to certify that at the close of business March 31, 1920, the balance in The Troy Trust Company to the credit of the American Water Works Association was Twenty-eight hundred sixty-eight and 32/100 Dollars ($2868.32).

Leland Law,

Secretary.

Permit me to submit the report of the financial condition of the Electrolysis Investigation Fund.

Contributions were received from thirty-nine sources as follows:

Cattlettsburg, Kenova & Ceredo Water Company.........	$10.00
City of Meadville, Pa..................................	10.00
Champaign and Urbana Water Company.................	25.00
Richmond, Ind., City Water Works......................	50.00
Portland, Me., Water District..........................	50.00
York, Pa., Water Company.............................	10.00

J. Waldo Smith, New York City, N. Y.	$10.00
Quincy, Ill., Water Works Commission	10.00
City of Colorado Springs, Colo	50.00
Kitchener, Ont., Water Commission	5.00
Acquackononk Water Co., Passaic, N. J	50.00
Passaic Water Company, Paterson, N. J	50.00
New Haven Water Company, New Haven, Conn	300.00
Hackensack Water Company, Weekawken, N. J	100.00
Davenport Water Company, Davenport, Iowa	100.00
Baltimore County Water and Electric Company	100.00
Elmira, N. Y., Water Board	10.00
Stamford Water Company, Stamford, Conn	250.00
Cortland, N. Y., Water Board	25.00
Charleston, S. C., Commissioners Public Works	25.00
Vinton-Roanoke Water Company, Roanoke, Va	5.00
Tampa Water Works, Tampa, Fla	50.00
Ansonia Water Company, Ansonia, Conn	25.00
Newport News Light and Water Company, Newport News, Va	50.00
Norristown Insurance and Water Company, Norristown, Pa	10.00
City of Baltimore, Md	200.00
Board of Water Commissioners, Glens Falls, N. Y	10.00
Bureau of Water Supply, Troy, N. Y	100.00
Kansas City, Mo., Water Department	50.00
Biddeford and Saco Water Company, Biddeford, Me	25.00
York County Water Company, Kennebuck, Me	10.00
Rensselaer Water Company, Rensselaer, N. Y	10.00
Springfield City Water Company, Springfield, Mo	25.00
Bureau of Water, Rochester, N. Y	200.00
Board of Water Commissioners, East Orange, N. J	100.00
Lexington Water Works, Lexington, Ky	25.00
Clear Springs Water Company, Bethelehem, Pa	25.00
Queens County Water Company, Far Rockaway, N. Y	50.00
Birmingham Water Company, Derby, Conn	25.00
Total	$2,235.00
Disbursements	532.68
Balance	$1,702.32

Attached you will find certificate from the Union National Bank showing a balance on deposit at the close of business March 31, 1920, of $1702.32.

Respectfully submitted,

JAMES M. CAIRD,

Treasurer.

This is to certify, That the amount standing to the credit of James M. Caird, Treasurer, American Water Works Association, at the close of business March 31, 1920 was Seventeen hundred two dollars thirty-two cents ($1702.32).

THE UNION NATIONAL BANK OF TROY,
JOSEPH E. KOBER,
Assistant Cashier.

SOCIETY AFFAIRS

THE ANNUAL CONVENTION

The Fortieth Annual Convention, at the Hotel Windsor, Montreal, P. Q., was formally opened on June 22, 1920, in accordance with the constitutional requirement that the first session shall be held on a Tuesday. A preliminary gathering was held on the evening of June 21, at which H. G. Hunter, chairman of the Local Entertainment Committee, presided. In opening this meeting, the chairman mentioned that Mayor Mederic Martin, who was present, had just been through an experience very rare in municipal affairs—a strike of the workmen in the water department lasting about thirty-six days.

His Worship, Mayor Martin, then welcomed the Association in an address outlining the importance of good water supplies and touching on the value of a convention of the Association to the water department of a city where it is held.

R. A. Ross, president of the Engineering Institute of Canada and a member of the Administrative Commission of the City of Montreal, welcomed the Association in the name of the engineers of the Dominion, and then gave a brief outline of the effect of the water works strike during the previous winter. The water stored in the reservoir of the municipal plant was soon exhausted, the supply to the higher parts of the city stopped, and some houses had to be deserted in consequence. At the time there was no snow to speak of on the ground and the frost had worked down 6 feet from the surface.

On New Year's morning a telephone call was sent to the engineers of the larger enterprises in the city and about fifty gathered to together. They were told that the city needed stationary engineers, firemen, mechanics and materials, and it was their duty to provide them in the emergency. They did so, and in addition put on overalls themselves. With their help the work of the water department was carried on until the strike was settled, something that might have been impossible without the assistance of the members of the Engineering Institute.

President Davis replied to the addresses and at the close of the meeting there was an informal reception by Mayor Martin, followed by dancing.

Morning session, June 22. Presiding officer, President Davis, who appointed T. A. Leisen, W. S. Cramer and C. R. Wood a committee on resolutions.

The committee appointed to canvass the ballots for officers reported the election of the following nominees: For president, Beekman C. Little; for vice-president, Dr. Edward Bartow; for treasurer, James M. Caird; for trustee, District 2, Harry F. Huy; for trustee, District 5, Robert J. Harding.

The Executive Committee reported that it had three amendments to the Constitution to present. The first, recommended by the Committee, related to the procedure to be followed by the Nominating Committee and was designed to eliminate the large expense of the committee's work under the Constitution as amended in 1918. After amendment, the proposed amendment was carried in the following form:

Amend Article VI, Section 4, to read as follows:

As the last order of business of the second session of the first day of the annual convention, the members from each District shall elect a member of the Nominating Committee to represent their respective Districts. Due notice of such election shall be prominently given in the program of the Convention, except in 1920, which shall be mailed to the members at least three weeks previous to the opening date of the convention. The votes of the Districts shall be by ballot or acclaim, and a majority vote of the members of each District present and voting shall elect the member of the Nominating Committee to represent that District. The members of the Nominating Committee so elected, together with the latest past-president present at the convention, who shall be Chairman, shall constitute the Nominating Committee to place in nomination candidates for the offices to be filled for the ensuing year.

Amend Article VI, Section 5, to read:

The Nominating Committee shall hold a meeting at 8.30 a.m. on the second day of the convention, previous to which time suggestions of names to fill the various offices may be made by the members of the Association to the members of the Nominating Committee, or by leaving same with the Secretary of the Association prior to the meeting of the Committee; names sent to the Secretary by mail at any time prior to the meeting of the Committee shall also be presented to the Committee for consideration. Nominations shall be by majority vote of the Nominating Committee, which must place in nomination

one candidate, and may place in nomination two candidates, for each office to be filled.

At any time prior to noon on the first day of March of each year additional nominations may be made by request to the Secretary, signed by at least 25 Active, Honorary or Corporate members, and upon receipt of such request the Secretary shall, after acceptance of the nomination by the candidate, add such additional nominees to the final ballot to be prepared by him. The nominees of the Nominating Committee shall head such final ballot for each office, and any additional nominees for the respective offices shall be placed under the nominees of the Committee in alphabetical order.

The second amendment, proposed to change the Secretary from an appointee of the Executive Committee to an officer elected by the Association. It had been submitted to the Executive Committee in accordance with the requirements of the Constitution, was submitted to the convention with the disapproval of the Executive Committee, and was not adopted.

The third amendment, recommended by the Executive Committee, eliminated Article V, Section 7, of the Constitution, to take effect April 1, 1921, and was accompanied by a memorandum that if the amendment was adopted notice should be sent to all organizations whose members were entitled to admission to the American Water Works Association without payment of an initiation fee, that this privilege would remain in force only until April 1, 1921. This amendment was carried.

The Annual Presidential Address[1] was read by President Davis, after which the following papers, illustrated by lantern slides, were presented, no discussion of them being permitted owing to the lateness of the hour:

"The Works of the Montreal Water and Power Company," F. H. Pitcher.

"The Municipal Water Supply of Montreal," Thomas W. LeSage.

Afternoon session, June 22. Presiding officer, President Davis.

The following papers were presented:

"Statistics of Water Supplies in the Province of Quebec," Theo. J. Lafreniere; discussion by John N. Chester, R. O. Wynne-Roberts, Carleton E. Davis and the author.

"Water Works of the City of Quebec," Charles P. Casgrain; discussion by Carleton E. Davis, Beekman C. Little and the author.

[1] See page 652 of this JOURNAL.

"Experiences in Montreal in the Manufacture of Alum," James O. Meadows; discussion by A. U. Sanderson, William Gore and the author.

The chair announced the election of the following members of the Nominating Committee: District 1, Theo. J. Lafreniere; District 2, George C. Andrews; District 3, Herman Rosenstreter; District 4, H. E. Keeler; District 5, E. L. Fulkerson; District 6, Thomas Maloney. Col. T. A. Leisen, the latest past-president at the convention, was chairman of the Committee, under the amendment adopted at the morning session.

Evening session, June 22. Presiding officer, President Davis. The papers presented were:

"Water Works Experiences,"[2] Beekman C. Little.

"Some Economic Features of Pumping Station Operation," Leonard A. Day, with lantern slides.

"Difficulties in Building the Louisville Pumping Station," James B. Wilson, with lantern slides.

Afternoon session, June 23. Presiding officer, President Davis. The papers presented were:

"The Basic Principles Used in the Designs for the New Water Supply Works for Winnipeg," James H. Fuertes, with lantern slides.

"Construction Features of the Greater Winnipeg Water District Soft Water Supply Scheme," W. G. Chase, with lantern slides.

Reports from the Illinois, Iowa and Minnesota Sections were presented by George C. Habermeyer, Jack J. Hinman, Jr., and F. W. Cappelen, respectively. During the year the Illinois Section held two meetings and the Iowa Section one. Both Sections endeavor to increase the membership in the Association in connection with the correspondence relating to the local meetings and in other ways. More interest is shown in the technical features of these meetings than in the social features. The Illinois Section has not done any work to better the public standing of water departments but the Iowa Section has an active legislative committee.[3] Neither Section has discussed any of the papers previously printed in the Journal. Informal discussions at the Illinois Section meetings are stimulated by the presiding officer calling upon the members for their views and experiences; in the Iowa Section a member is gen-

[2] See page 641 of this Journal.

[3] See Journal, March, 1920, page 152.

erally requested in advance to be ready to open the discussion on each of these topics. Occasional joint meetings with Sections in adjoining districts were favored by the Illinois and Iowa Sections.

Mr. Cappelen reported for the Minnesota Section that it is progressing nicely and had held several meetings during the year. He considered his most important report to be, however, that although this Section had won the Hill Cup for the largest percentage of increase in membership during three successive years, 1917, 1918 and 1919, the members of the Section considered the effect of war conditions on membership so disturbing that it was unfair to accept the prize and it was accordingly returned for further competition.[4]

The report of the Finance Committee[5] was presented by Col. George A. Johnson, chairman, and adopted by formal vote. The report recommended an increase in annual dues, which can only be made by amending the Constitution; in order that the Executive Committee might have full knowledge of the views of all members on the subject, made important by the substantial increase in committee work of the Association and a general increase in its expenses, it was voted to take a letter ballot of the entire membership of the Association upon the advisability of increasing the annual dues of active members from $5 to $7, those of corporate membership from $5 to $10 and those of associate members from $10 to $15, these dues to go into effect on April 1, 1921. Colonel Johnson explained that while the annual report of the Finance Committee recommended an increase in the annual dues of active members from $5 to $6, the work of the committees of the Association would be materially assisted if the dues were made $7 and the Committee preferred to see them raised to that amount.

The report of the Publication Committee[6] was read by title.

The report of the Association's representatives on the American Electrolysis Committee[7] was presented by E. E. Minor. In addition to presenting the report Mr. Minor stated that the Committee was inaugurating extensive field investigations at East St. Louis, Ill., which were expected to furnish valuable information on

[4] Later in the Convention it was announced that the cup had been won for 1919–1920 by the Canadian Section.

[5] See JOURNAL, July, 1920, page 622.

[6] See JOURNAL, July, 1920, page 628.

[7] See JOURNAL, July, 1920, page 618.

the effect of electrical drainage of pipe systems on electrolytic corrosion at joints and related phenomena. He also presented a report[8] from the Bureau of Standards and another from Prof. L. A. Hazeltine, the Association's expert assisting its representatives on the American Electrolysis Committee, approving the practice of grounding the secondaries of electric lighting transformers to water pipes, providing the grounding is done in accordance with the National Electrical Code and where several buildings are supplied from one transformer or group of transformers, but one ground connection is made to the water piping.

The report[9] of the Committee on Cold Weather Troubles was presented by Charles R. Bettes, chairman.

The following resolution, introduced by Colonel Johnson, was carried after some discussion:

WHEREAS, the American Water Works Association, in convention assembled, being mindful of its responsibilities as the national representative of water works men and realizing that such questions as methods of analysis of water, oil, coal and other minerals and metals encountered in the operation and management of water works plants and systems properly should be the subject of research and standardization by the American Water Works Association; and

WHEREAS, the establishment of reasonable standards of satisfactory water for domestic and various industrial uses is a desideratum in the field of public sanitation, as are standards respecting the kind and extent of purification processes to be adopted for the satisfactory correction of various types of raw water as well as the design and equipment of purification plants and systems; and

WHEREAS, the American Water Works Association counts among its members many practical chemists, bacteriologists, epidemiologists, statisticians, consulting engineers and practical operators of water purification plants;

Therefore Be It Resolved, that a Council on Standardization be formed; that this Council consist of five active members of the Association appointed by the President; and that the Council in turn appoint for the purpose of research and report certain sub-committees whose specific duties will be:

1. To conduct the necessary research and promulgate standard methods of chemical and bacteriological analysis of water, and standard methods of analysis of coal, oil, chemicals, metals, cement, sand and other supplies employed in water works construction and operation. In this connection it will be the purpose of this Association to coöperate with similar committees of other professional societies with a view to unification of methods.

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[8] See page 761 of this JOURNAL.

[9] See page 749 of this JOURNAL.

2. To investigate and promulgate standards of satisfactory quality of water, with particular regard to the establishment of reasonable criteria respecting what analytical units constitute a pure and wholesome water for domestic consumption; a satisfactory water for the several more important industrial uses; and the kind and extent of purification processes to be adopted for the correction of pollution.

3. To standardize, so far as practicable, the design and equipment of purification plants and systems, and methods of recording operating and analytical results therefrom.

The vote for the city in which to hold the next convention resulted in the recommendation of Cleveland, Ohio, to the Convention Committee of the Association.

Morning session, June 24. Presiding officers, President Davis and Trustee Cramer.

The Nominating Committee reported that at a meeting at which all members but one were present, the following nominations were made unanimously: For President, Dr. Edward Bartow; for Vice-President, W. S. Cramer; for Treasurer, William W. Brush; for Trustee, Third District, George C. Gensheimer; for Trustee, Sixth District, J. Chris. Jensen.

The following resolution, recommended by the Executive Committee, was passed, subject to its final approval by the National Board of Fire Underwriters:

Whereas, the grounding of the secondaries of lighting transformers to water pipes promotes safeguarding of life and does not constitute a hazard to the piping system, be it

Resolved, that the American Water Works Association approves the practice of grounding the secondaries of lighting transformers to the water pipes, providing that the grounding is done in accordance with the rules of the National Electrical Code and the National Safety Code.

The question, "What is the Proper Size of Meter for Multiple-Family Houses," was discussed by Henry P. Bohmann, G. A. Elliott (by letter), D. W. French, W. R. Edwards, F. T. Kemble, A. W. Cuddeback, J. N. Chester, J. Walter Ackerman, W. Z. Smith, A. H. Kneen, Wm. Luscombe, Adolph Mueller, Wm. A. Nial, E. E. Davis, Wm. F. Sullivan, and William W. Brush, and the convention voted to request the President to appoint a committee to investigate the subject.

A paper on "Standardization of Brass Goods for Water Works" was read by Adolph Mueller and discussed by J. M. Diven, J. N.

Chester and the author. The convention voted to request the President to appoint a committee to coöperate with similar committees of the New England Water Works Association and of the national organization of manufacturers of brass goods in preparing standards for brass goods for water works.

The Committee on Standardization of Specifications for Water Meters presented a progress report.

Afternoon session, June 24. Presiding officers, President Davis and Trustee Cramer.

A paper on "The Prevention of Water Waste on Railroads" was presented by C. R. Knowles and discussed by Charles Haydock.

An informal address on "Ice Formation," illustrated by lantern slides and moving pictures, was made by invitation by John Murphy, electrical engineer of the Canadian Department of Railways and Canals. It explained the nature of anchor ice and frazil and outlined methods of preventing their formation. The subject was discussed by Charles H. Lord, H. F. Dunham, F. H. Pitcher, Wm. W. Brush, A. U. Sanderson, C. A. Wadsworth, C. Arthur Brown and Mr. Murphy.

A paper on "Some Aspects of Electrolysis" was presented by Dr. Gellert Alleman and discussed by H. F. Dunham, L. C. Whitsit, N. McL. R. Wilson, V. Bernard Siems and the author.

It was voted to be the sense of the convention that a public expression of appreciation should be made for the faithful work done by Secretary Diven in the past and that the Executive Committee should retain him in office.

Secretary Diven announced the following gains in membership of the different Sections during 1919–1920; California, 17 members, an increase of 46 per cent; Central States, 45 members, an increase of 30 per cent; 4-States, 31 members, an increase of 22 per cent; Illinois, 17 members, an increase of 15 per cent; Iowa, 17 members, an increase of 40 per cent; Minnesota, 30 members, an increase of 42 per cent; New York, 34 members, an increase of 29 per cent; Canada, 49 members, an increase of 79 per cent, winning the Hill Cup for 1919–1920, as previously stated.

Evening session, June 24. Presiding officer, Trustee Cramer.

A paper on "The Revenue Chargeable to Public Uses of Water in the City of Rochester" was presented by Stephen B. Story and discussed by Leonard Metcalf, Walter E. Miller and the author.

A paper on "Damage to Deep Wells by Sea Water," with lantern-slide illustrations, was presented by Dr. Wm. P. Mason and discussed by H. F. Dunham, William Gore, Chester B. McFarland, Dr. Rudolph Hering and the author.

A paper on "Cost-Plus Contracts in Water Works Construction" by George W. Fuller was presented by title, owing to the lateness of the hour and the absence of the author.

A report[10] by the Associated General Contractors of America on "Standard Forms for Contracts" was presented by invitation by G. W. Buchholz and discussed by Wm. Luscombe, J. M. Diven and J. M. Goodell. It was stated in the discussion that under Article VIII, Section 1, the President had authority to appoint a special committee to act with the special committee of the Associated General Contractors of America and therefore no vote to authorize this proceeding was needed.

A paper[11] on the "War Burden of Water Works in the United States Continues" was presented by Leonard Metcalf and discussed by J. M. Diven, W. E. Miller, W. R. Conard, A. P. Folwell and the author.

On motion of Past-President Leisen, chairman of the Resolutions Committee, the following resolution was adopted:

Resolved: That the American Water Works Association records its deep appreciation of the untiring and highly successful efforts of Messrs. Hunter and Hutchinson, all other members of the Entertainment Committee of Montreal, and their associates in providing for the pleasure and entertainment of the members and guests of the Association during their sojourn in this city.

There were about 310 active members, 150 associate members and 225 guests at the convention. On the morning of June 23 there was a trip by steamer through the Lachine Rapids, in the evening of that day a smoker, and on the morning of June 25 a visit to the water purification plants of the Montreal Water Department and of the Montreal Water & Power Company, luncheon being served at the latter. Excursions about the city, theatre and card parties were provided for the ladies during the convention.

Chemical and bacteriological section, morning session, June 24. Presiding officer, Dr. Edward Bartow. The following papers were presented:

[10] See page 657 of this JOURNAL.
[11] See JOURNAL, July, 1920, page 471.

"Standards of Quality of Water," Jack J. Hinman, Jr.; discussed by Dr. Rudolph Hering, William Gore, Norman J. Howard, William J. Orchard, Richard Messer, Dr. Bartow and the author.

"Coöperative Research in Water Purification,"[12] Abel Wolman; discussed by Sheppard T. Powell, Robert Spurr Weston and Wm. J. Orchard. The Section voted to endorse the recommendations in the paper and to request the Council on Standardization to investigate the possibility of coöperative research before the next convention.

"Index Numbers and the Scoring of Water Supplies," Abel Wolman; presented by title.

"Recent Progress in the Fight against Typhoid Fever," Wm. J. Orchard; discussed by Dr. W. P. Mason, Leonard Metcalf, Abel Wolman, Richard Messer, Helman Rosenthal, Jack J. Hinman, Jr., Dr. Bartow and the author.

Chemical and bacteriological section, afternoon session, June 24. Presiding officer, Robert Spurr Weston. The following papers were presented:

"A Mailing-Case Method of Long-Distance Bacteriological Control of Water Supplies," MacHarvey McCrady; discussion by Abel Wolman and the author.

"The Dallas, Texas, Water Purification Plant," Helman Rosenthal.

The following officers of the Section were elected: For Chairman, Capt. Jack J. Hinman, Jr.; for Secretary, Abel Wolman; for Executive Committee, MacHarvey McCrady, Wm. J. Orchard and J. C. Diggs.

The attendance at the sessions of the Section was about fifty.

BACK COPIES OF THE JOURNAL WANTED

The rapid growth of the Association this year has practically exhausted the supply of JOURNALS for January, March and May, 1920, in the Secretary's office. Members who have copies of these issues which they do not care to preserve are requested to send them to the Secretary, at 153 West 71st Street, New York.

[12] See JOURNAL, July, 1920, page 572.

ERRATA

In the paper on "Restricting Waste of Water in San Francisco," in the JOURNAL for May, 1920, the following corrections should be made on page 293. Middle column of table, change "gallons" to cubic feet. Same change should be made in second line of the paragraph immediately below the table, where "4000 cubic feet" should be substituted for "4000 gallons."

In the paper on "Electric Cast-Iron Pipe and Prepared or Factory-Made Lead Joints" in the JOURNAL for July, 1920, the following corrections should be made: Page 479, cut out the words "Mandrel used in making the joints" in figure 2. Page 485, line 7, change "above" to "below," and in the next to the last column of the table, change "lbs." to "lbs. per sq. in."

NEXT CONVENTION

The Convention Committee has selected the Hollenden Hotel at Cleveland, Ohio, as headquarters for the next annual convention and fixed the date as the week of June 6, 1921. Some objections to this date have been raised by members connected with educational institutions, as they will not be able to attend a convention held before the close of the school year, which is later than June 6, as a rule.